Factors Affecting the Behaviour of the Shear Connection of Steel-Concrete Composite Beams

by

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Statement of Authentication

The work presented in this thesis is, to the best of my knowledge and belief, original except as acknowledged in the text. I hereby declare that I have not submitted this material, either in full or in part, for a degree at this or any other institutions

.................................................................

(Signature)
Preface

This thesis is submitted to the University of Western Sydney, Australia, for the degree of Doctor of Philosophy. The theoretical and most of the experimental work described in this thesis was carried out by the candidate during the years 2003-2006 at the Centre for Construction Technology and Research under supervision of Dr Mark Patrick and, after its closure in 2004, at the Construction Technology and Research Group as part of the School of Engineering under the combined supervision of Dr Andrew Wheeler and Emeritus Professor Russell Q. Bridge. However, some of the experimental work presented in Chapters 4 and 5 had already been conducted at the Centre for Construction Technology and Research prior the commencement of the author’s candidature.

Although limited in number by the confidential nature of some of the reinforcing products investigated, several conference and journal papers were written in conjunction with the supervisors on the work presented in this thesis. These papers are:


Several more publications on this research work are in preparation and will be submitted for review shortly:

Ernst, S., Bridge, R. Q., and Wheeler, A. "Push-out tests and a new approach for the design of secondary composite beam shear connections." *Journal Construct. Steel Research*


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**NOTATION**

Wherever applicable, the symbols used are chosen to be consistent with the symbols given in AS2327.1 (Standards Australia 2003a). In the following, the most frequently used notations are defined. The definitions of other symbols are given in the relevant section.

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<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
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<tr>
<td>$A_{sc}$</td>
<td>cross-sectional area of the shank of the headed stud</td>
</tr>
<tr>
<td>$A_{cone}$</td>
<td>projected horizontal failure cone surface of shear connections experiencing stud pull-out failure</td>
</tr>
<tr>
<td>$A_{side}, A_{top}$</td>
<td>surface areas of failed concrete wedge</td>
</tr>
<tr>
<td>$A_{sh}$</td>
<td>cross-sectional area of concrete haunch reinforcement per unit length</td>
</tr>
<tr>
<td>$A_{sh,h}$</td>
<td>cross-sectional area of horizontal transverse concrete haunch reinforcement containing longitudinal splitting failures</td>
</tr>
<tr>
<td>$A_{sh,v}$</td>
<td>cross-sectional area of vertical concrete haunch reinforcement crossing the horizontal haunch shearing failure surface</td>
</tr>
<tr>
<td>$A_{sv}$</td>
<td>cross-sectional area of longitudinal waveform reinforcement crossing a horizontal failure surface</td>
</tr>
<tr>
<td>$b_{cf}$</td>
<td>width of concrete slab compression flange</td>
</tr>
<tr>
<td>$b_{cone}$</td>
<td>width of concrete cone after stud pull-out failure</td>
</tr>
<tr>
<td>$b_{ct}$</td>
<td>width of concrete rib at mid-height of the steel ribs</td>
</tr>
<tr>
<td>$b_{ctr}$</td>
<td>width of concrete rib between top edges of the steel ribs</td>
</tr>
<tr>
<td>$b_{eff}$</td>
<td>effective transverse width of stud pull-out or rib shearing failure surface</td>
</tr>
<tr>
<td>$b_{esh}$</td>
<td>effective transverse width of over which steel sheeting is assumed to contribute to rib punch-through capacity</td>
</tr>
<tr>
<td>$b_{ewb}, b_{ewt}$</td>
<td>effective width of rib punch-through concrete wedge at bottom and top of concrete rib</td>
</tr>
<tr>
<td>$b_{h}$</td>
<td>width of concrete haunch measured at bottom of haunch</td>
</tr>
<tr>
<td>$c_1, c_2, c_{1m}, c_{2m}$</td>
<td>calibration factors for solid slab shear connection strength (see Table 2.1 and Table 6.1)</td>
</tr>
<tr>
<td>$c_b$</td>
<td>horizontal distance of reinforcement bars from the edge of the concrete rib</td>
</tr>
</tbody>
</table>
\( d_{bs} \) = nominal stud diameter of a headed stud
\( d_{br} \) = nominal diameter of reinforcement bar
\( D_c \) = overall thickness of concrete slab
\( d_{ed} \) = nominal diameter of stud performance-enhancing device
\( e \) = longitudinal distance from the shear connection to the edge of the concrete rib at mid-height in the concrete bearing zone direction
\( e_b, e_t \) = longitudinal distance from the shear connection to the bottom and top edge of the concrete rib
\( e_{crit} \) = critical longitudinal distance of peak tensile stress from the shear connection at which the concrete wedge breaks out
\( E_c \) = elastic modulus of slab concrete
\( E_{sh} \) = elastic modulus of steel sheeting
\( f_c \) = 28 day characteristic compressive cylinder strength of concrete
\( f_{cm} \) = average compressive strength of sample cylinders cured alongside test specimens
\( f_{ds} \) = design shear capacity of a shear connector
\( f_t \) = tensile strength of concrete
\( f_{uc} \) = tensile strength of shear connector material
\( f_{vs} \) = nominal shear capacity of a shear connector
\( f_{vs,e} \) = experimental capacity of a shear connector
\( f_{vs,m} \) = mean shear capacity of a shear connector
\( f_{yr} \) = yield strength of steel reinforcement
\( f_{ysh} \) = yield strength of steel sheeting
\( G_{nm}, G_n \) = mean and nominal dead loads
\( h_c \) = overall height of shear connector
\( h_{ec} \) = effective stud height over which the shear forces are transferred into the surrounding concrete
\( h_{ed} \) = effective height of stud enhancing device over which the shear forces are transferred into the surrounding concrete
\( h_t \) = height of concrete rib
\( k_{bs} \) = correction factor considering the beneficial effects of transverse reinforcement in concrete ribs
\( k_{ec} \) = correction factor considering the embedment depth of the stud into the cover slab
\( k_t \) = reduction factor for determining the nominal strength of secondary composite beam shear connections
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<thead>
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<th>Description</th>
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<tr>
<td>$k_{RPT,max}$</td>
<td>correction factors for the various failure modes to apply a uniform design resistance factor $\phi = \phi_{\text{Solid}}$</td>
</tr>
<tr>
<td>$k_{RPT,min}, k_{RS/SP}$</td>
<td>reduction factor considering the non-linear stress distribution along the proposed rib shearing/stud pull-out failure surface</td>
</tr>
<tr>
<td>$k_\sigma$</td>
<td>length between top and bottom corner of steel decking rib</td>
</tr>
<tr>
<td>$L_m, L_n$</td>
<td>mean and nominal load effects</td>
</tr>
<tr>
<td>$n_x$</td>
<td>number of shear connectors in a group at a transverse cross-section of a composite beam</td>
</tr>
<tr>
<td>$P_{\text{wed}}$</td>
<td>Capacity of a concrete rib against a wedge break-out</td>
</tr>
<tr>
<td>$P_{\text{wed,bs}}$</td>
<td>Capacity of a concrete rib including transverse rib reinforcement against a wedge break-out</td>
</tr>
<tr>
<td>$P_{RPT,max}$</td>
<td>stud capacity of a shear connection against rib punch-through failure</td>
</tr>
<tr>
<td>$P_{RPT,min}$</td>
<td>stud capacity which can be guaranteed at any given slip up to the required slip capacity for a shear connection experiencing rib punch-through failure</td>
</tr>
<tr>
<td>$P_{\text{RS}}$</td>
<td>stud capacity of a shear connection against rib shearing failure</td>
</tr>
<tr>
<td>$P_{\text{sh}}$</td>
<td>force component transferred by the steel decking</td>
</tr>
<tr>
<td>$p_{\text{sh,b}}$</td>
<td>load component transferred by the steel decking bending mechanism</td>
</tr>
<tr>
<td>$p_{\text{sh,t}}$</td>
<td>load component transferred by the steel decking tensile mechanism</td>
</tr>
<tr>
<td>$P_{\text{Solid}}$</td>
<td>stud capacity of a shear connection in a solid slab</td>
</tr>
<tr>
<td>$P_{SP}$</td>
<td>stud capacity of a shear connection against stud pull-out failure</td>
</tr>
<tr>
<td>$Q_{m}, Q_n$</td>
<td>mean and nominal live loads</td>
</tr>
<tr>
<td>$R_m, R_n$</td>
<td>mean and nominal resistance</td>
</tr>
<tr>
<td>$R_i$</td>
<td>theoretical strength function used to determine the resistance</td>
</tr>
<tr>
<td>$s_c$</td>
<td>longitudinal spacing of shear connectors between adjacent groups</td>
</tr>
<tr>
<td>$s_x$</td>
<td>transverse centre-to-centre spacing of shear connectors in a concrete rib</td>
</tr>
<tr>
<td>$t$</td>
<td>thickness of steel sheeting</td>
</tr>
<tr>
<td>$T_c$</td>
<td>tensile forces acting in the shank of a shear connector</td>
</tr>
<tr>
<td>$T_{\text{sh}}$</td>
<td>longitudinal tensile forces in the steel decking</td>
</tr>
<tr>
<td>$T_{yc}$</td>
<td>tensile capacity of the shank of a shear connector</td>
</tr>
<tr>
<td>$u_{hs}$</td>
<td>perimeter length of haunch shearing failure surface</td>
</tr>
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<td>Description</td>
</tr>
<tr>
<td>------------</td>
<td>-----------------------------------------------------------------------------</td>
</tr>
<tr>
<td>$V$</td>
<td>coefficient of variation, subscripts are the variables studied</td>
</tr>
<tr>
<td>$V_L$</td>
<td>longitudinal shear capacity per unit length of the concrete slab</td>
</tr>
<tr>
<td>$\alpha$</td>
<td>angle of failed concrete wedge to the transverse of the steel beam</td>
</tr>
<tr>
<td>$\alpha_s$</td>
<td>inclination of reinforcement towards the vertical line</td>
</tr>
<tr>
<td>$\beta$</td>
<td>reliability index</td>
</tr>
<tr>
<td>$\beta_0$</td>
<td>target reliability index</td>
</tr>
<tr>
<td>$\delta_c$</td>
<td>longitudinal slip of a shear connection</td>
</tr>
<tr>
<td>$\delta_{sh}$</td>
<td>mid-span deflection of steel decking in concrete rib</td>
</tr>
<tr>
<td>$\gamma_G$, $\gamma_Q$</td>
<td>load factors for dead and life load</td>
</tr>
<tr>
<td>$\phi$</td>
<td>resistance factor for a strength limit state</td>
</tr>
<tr>
<td>$\phi_{Solid}$</td>
<td>resistance factor for a strength limit state for stud shearing failure</td>
</tr>
<tr>
<td>$\rho_c$</td>
<td>density of concrete</td>
</tr>
<tr>
<td>$\theta$</td>
<td>angle between the steel ribs of a composite slab and the longitudinal axis of a steel beam</td>
</tr>
<tr>
<td>$\theta_{sh}$</td>
<td>rotation of steel decking web</td>
</tr>
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ABSTRACT

The inclusion of trapezoidal types of steel decking in the shear connection of composite beams has been found to significantly reduce their maximum strength and ductility by causing premature concrete-related failure modes. In order to investigate the complex behaviour and various load-transfer mechanisms that can occur in composite beams incorporating this type of connections, a total of 91 carefully-designed push-out tests were performed. Specific failure modes in conventionally reinforced specimens were initially induced by varying critical parameters. Specimens incorporating specific stud reinforcing devices were subsequently tested to suppress the undesirable failure modes. The concrete-reinforcing and stud performance-enhancing devices, which included novel waveform-type reinforcement elements and spiral wire or ring components surrounding individual studs in secondary composite beams and special haunch reinforcement in primary beam applications, significantly delayed the onset and reduced the effect of the premature concrete-related failure modes. Hence, they increased the ultimate strength and ductility of the shear connection. The findings of the small-scale push-out tests were also verified in two full-scale composite beam tests which showed good agreement in shear connection behaviour and failure mode.

Most of the design approaches currently used around the world take into account the weakening effect of trapezoidal types of decking by applying a reduction factor to the nominal strength that the same connection would have in a solid slab. From the test results, it is evident that not every shear connection incorporating steel decking, and within the limits of the associated standards, can be classified as ductile. A new and more reliable design approach is proposed which also incorporates the application of the various stud reinforcing devices. The key element of this design approach is to classify the anticipated connection behaviour, in regards to its deformation capacity, into ductile or brittle connections, hence ensuring satisfactory shear connection behaviour where the new types of trapezoidal steel decking are used. A reliability analysis of the new proposal is presented which enables the application of this new approach in accordance with AS 2327.1 (Standards Australia 2003). It is calibrated to provide a reliability index similar to stud applications currently in use. Simple strength reduction factors for the types of trapezoidal steel decking available in Australia are also provided which can be applied to the current solid slab shear connection strength for a fast and simplified design.
1 INTRODUCTION

1.1 Structure of the shear connection in composite beams

Composite construction in steel and concrete has become more and more popular in developed countries in recent years. It combines the advantages of the two materials, steel and concrete, very efficiently and leads to new possibilities in the design of multi-storey and industrial buildings and bridges.

In a composite beam subject to bending, the concrete slab and the steel beam are connected by a shear connection in the longitudinal direction along their interface. The connection ensures composite action and, as a result, forms one structural element out of the two different elements. Composite action significantly improves the behaviour of the beam for both the serviceability and the ultimate limit state. For example, the flexural stiffness and the moment capacity in a composite beam are considerably increased compared to a bare steel beam. Hence, composite beams are known to form a very cost-effective structural element, widely used in various applications.

The shear connection between the steel beam and the concrete slab is the crucial component to develop composite action. It needs to resist longitudinal slip and separation forces at the interface of both structural elements. The shear connection of a composite beam consists of several components which can all have a significant impact on the overall behaviour of the connection (Figure 1-1):
The shear connectors are the core component of the connection. They are usually attached to the top flange of the steel beam. There are numerous types of mechanical shear connectors available worldwide. Some of them are patented by the manufacturer and their design is accounted for in product-specific guidelines. Examples are the HILTI X-HVB shear connector or the cold-formed steel “StripCon” connector (Fontana and Bärtschi 2002). However, in most cases freely available constructional steel elements are utilized as shear connectors and their design is accounted for in the applicable standards. Currently, three different types of shear connectors are defined in the Australian Standard 2327.1: Composite Structures, Part 1: Simply Supported Beams (Standards Australia 2003a). They are: (a) welded headed stud connectors, (b) steel channel connectors and (c) high-strength structural bolts (Figure 1-2). Of these, headed studs are by far the most popular shear connector type with an application rate of well over 90% in the construction of composite beams. They are cheap, easy to attach to a steel section and provide a high individual shear capacity and ductility. Taking into consideration the dominant position of the headed stud as a shear connector, this research solely investigates this connector type. The test programs and design provisions derived herein were all adjusted to the stud connector. Nevertheless, many of the findings presented herein can also be applied to other types of shear connectors.

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The top flange of the steel beam may be subject to significant compressive forces, particularly where partial shear connection is used. Buckling of the top flange...
between individual shear connectors or local rotation of the section due to the restraining moments imposed by the connectors are possible weakening effects that slender top flanges can impose on shear connections. These effects are already well accounted for in AS2327.1 (Standards Australia 2003a) by classifying the steel sections in compression into slender, compact and non-compact sections. The use of slender sections is prohibited while the effective area of non-compact sections is reduced to an equivalent compact size. Limits on the minimum thickness of the steel section top flange and the position of the shear connector are also given. In the case of a headed stud shear connector, for example, a minimum flange thickness of 0.4 times the stud thickness and a minimum clear distance of 20mm between the stud and the closest transverse edge of the flange are required. Hence, the influence of the steel beam top flange is not further investigated in the course of this thesis. However, one should be aware of the top flange being a part of the shear connection and its potential impact on the strength of the connection, particularly when novel types of shear connectors are used.

The transfer of concentrated loads into the concrete slab using distinct shear connectors leads to high local concrete stresses in the vicinity of the connectors which can cause concrete-related failure modes. The strength and the properties of the concrete are therefore significant factors in the resistance of a shear connection. In modern building structures, the use of structural steel decking as part of a composite slab is a popular feature. The structural steel decking is used as formwork during the construction stages and as tensile slab reinforcement when the construction is completed. Depending on its geometry, it may also significantly reduce the amount of concrete required for the slab. In composite slabs, the steel decking becomes an integral part of the shear connection and may have a significant impact on the behaviour of the shear connection as it can interrupt the flow of force. The application of steel decking geometries with closed or narrow open (dovetailed) steel ribs are already accounted for in AS2327.1 (Standards Australia 2003a), but profiles with trapezoidal wide-open ribs, which only recently have started to be manufactured for the Australia market, are currently not covered. At the same time, the influence of this type of steel decking on the shear connection behaviour can be most detrimental. Hence, testing and research described herein focuses mainly on these types of steel decking geometry.
In the most common case of a composite beam subject to positive bending, an effective area of the concrete slab is utilized as the composite beam compression zone. When the concentrated shear forces introduced by the shear connectors are distributed over the width of the effective concrete section, transverse tensile stresses are created in the slab which may create several types of longitudinal shear failures. In order to prevent these types of failures from occurring and to accommodate these tensile stresses, transverse steel reinforcement which crosses the potential longitudinal shear surfaces in the concrete slab forms an essential part of the shear connection. Negative bending moments in a composite beam require additional longitudinal reinforcement to resist the tensile stresses in the concrete slab. Special types of reinforcement or reinforcing devices may also be required to suppress or minimize the effect of certain kinds of unwanted concrete related failure modes in the vicinity of the shear connection as shown in the further course of this thesis.

1.2 Weakening effects of profiled steel decking

If flexible shear connectors such as headed studs are used, most of the shear forces are transferred at the base of the connector into the surrounding concrete which, in return, is subject to high local bearing stresses. Profiled steel decking typically reduces the amount of concrete surrounding the shear connectors in this region. Its use also creates additional free edges in the concrete slab. From fastener applications, it is known that free edges in the vicinity of an anchor reduce its strength if subject to longitudinal shear. Moreover, the presence of the steel decking creates voids which interrupt the natural flow of the longitudinal shear forces and induce new types of potential failure surfaces.

In particular, where trapezoidal types of steel decking are used, the shear connection is known to be prone to concrete-related failure modes. In headed stud shear connector applications, such failures have been found to appear at much lower levels of load than the same connector would experience in a solid concrete slab application. Hence, the shear connector itself is not utilized to its full potential capacity and these failure modes are classified as premature. The weakening effects
are widely acknowledged and the shear connector strength typically needs to be reduced when used in conjunction with trapezoidal types of steel decking, although the particular reduction factors commonly used have come into dispute in recent years and may require further revision. However, the fact that the different failure modes also lead to altered shear connection behaviour and lower slip capacity of the connection is usually overlooked. Figure 1-3 shows typical load-slip behaviours of headed stud shear connections experiencing ductile and brittle failure modes and the ideal-plastic design approximation used in the shear connection design for composite beams. The ideal-plastic approximation is only justified for the case of a ductile shear connection behaviour, i.e. where a sufficiently high slip capacity can be ensured, but it does not reflect the behaviour of some of the shear connections in composite slabs which experience relatively brittle behaviours, i.e. reduced slip capacities, regardless of how much the permissible strength may be reduced. Brittle shear connection behaviour similar to the one presented in Figure 1-3 is not acceptable where the composite beam design is based on the assumption of a ductile shear connection which allows for redistribution of connector forces within the composite beam.

![Figure 1-3: Actual and assumed load-slip behaviour of the headed stud shear connection](image)

**1.3 Measures to improve shear connection behaviour**

The lack of ductility in most of the premature failures originates in the development of concrete cracks which are able to form along unreinforced failure surfaces. As the
behaviour of concrete in tension is very brittle, the shear connection can not usually develop sufficient ductility if cracks occur. In contrast to solid slab shear connections, conventionally reinforced connections in composite beams incorporating steel decking usually do not comprise reinforcement placed in the vicinity of the shear connection base where, for flexible shear connectors, most of the longitudinal shear forces are transferred into the concrete slab as the concrete ribs typically remain unreinforced. Neither do these connections contain vertical reinforcement preventing potential horizontal shear failure surfaces forming between the edges of concrete ribs created by the use of the profiled steel decking geometry.

Consequently, an effective measure to improve the ductility is to reinforce the shear connection along potential crack surfaces as the reinforced concrete is able to ensure continuous load transfer after cracking occurs. For this purpose, a range of specific reinforcing products have been developed in Australia in recent years. In secondary edge beam applications, i.e. where the steel decking runs transverse to the steel section, close to a free concrete edge, a Type 4 horizontal shear failure already needs to be considered in accordance with AS 2327.1 (Standards Australia 2003a). To prevent this failure, a special waveform reinforcement element is widely available on the Australian market (Figure 1-4a). Even where the concrete slabs are wider, this waveform type of reinforcement was found to improve the behaviour of the shear connection significantly as it can reinforce against so-called stud pull-out failures. In primary composite beam applications, i.e. where the steel decking runs parallel to the
steel section, concrete haunches are formed. The application of special types of haunch reinforcement (Figure 1-4b) was found to prevent horizontal shearing failure of the concrete rib and overcame the effects of longitudinal splitting failure of the haunch.

Furthermore, two provisional patent applications have been lodged for a new stud performance-enhancing device (Patrick 2002a; Patrick 2002b) whose purpose is to confine the concrete around the base of the stud at all stages of loading, thus reducing bending of the stud shank, and minimising the effects of localized failures where the shear connectors are placed in the vicinity of a free concrete edge. One embodiment comprises a ring cut from a welded steel tube which surrounds the stud at its base in order to limit the effect of rib punch-through failure when secondary beams incorporating trapezoidal types of steel decking are used (Figure 1-5). The ring can be readily clipped into position over a stud once it has been welded to the beam flange.

**Figure 1-5:** Early prototype of stud performance enhancing device StudRing™

### 1.4 Aim and structure of the thesis

The aim of this thesis is to gain a thorough understanding of the various premature concrete related failure modes when profiled steel decking, in particular with wide-open ribs, is used. It is already evident that not every headed stud shear connection incorporating steel decking within the limits of the associated standards can be classified as ductile. A clear differentiation between shear connections with sufficient and insufficient ductility is therefore needed. Over the past few years, it was also
established by various researchers, e.g. Kuhlmann and Raichle (2006); Rambo-Roddenberry (2002); Yuan (1996), that the reduction factor approach for shear connections incorporating composite slabs does not provide reliable stud strengths for the existing generation of open rib steel decks, let alone new decks which are beginning to appear on the market. A new approach is needed in this area as well.

An important measure in achieving sufficiently ductile behaviours and more reliable stud strengths can be the application of the novel reinforcing devices, as described above, which have been found to be able to suppress the unwanted premature concrete-related failures where composite slabs are used. As a consequence, the overall connection strength might be increased and the ductility of the shear connection improved up to a level where an ideal-plastic assumption of the shear connection behaviour would be justified. However, the potential application of the various components requires a more thorough investigation so as to obtain a more detailed understanding of their mechanical behaviour.

Firstly, an extensive review of the previous research on headed stud shear connections in both solid and composite slab applications is undertaken in Chapter 2. In this, an emphasis is placed on the various models developed to take into account the altered shear connection behaviour in composite slab applications and on reinforcing components previously provided to improve behaviour. Information concerning the slip requirements of composite beam shear connections and test methods to determine its behaviour are also given. Generally, small-scale so-called push-out tests are used to derive the strength and deformation capacity of the shear connection. However, there is a large variety in the push-out test methods used among the various researchers. In Chapter 3 the influence of the different test methods on the overall behaviour of the shear connection are investigated and the test methods used for the subsequent push-out tests are described. During the course of this research, it was found that some of the push-out specimens tested did not behave in a satisfactory manner and experienced failures unrelated to the shear connection behaviour. This triggered the development of a novel horizontal push-test rig to overcome these problems and provide a larger range of potential applications. This is introduced as part of Chapter 3.
In the further course of the thesis, a systematic investigation into the effects of concrete related failure modes on the shear connection behaviour and into the application of reinforcing measures to subsequently overcome brittle shear connection responses is undertaken. For the most part, the investigation comprises an extensive series of small-scale push-out tests which are described in Chapter 4 for secondary beam applications and in Chapter 5 for primary beam applications. In these, the different failure modes experienced are identified and the test results are analysed with regards to the various factors that influence the shear connection behaviour. Additionally, a few carefully designed full-scale composite beam tests were performed in order to validate the findings of the push-out tests. The results are presented and evaluated in Chapter 7.

Numerous strength provisions for shear connectors in secondary composite beam applications exist, some of which were introduced in the last ten years to provide a better estimate that accounted for the various failure modes. The application of some of these methods to shear connections incorporating the new Australian types of trapezoidal steel decking is critically reviewed in Chapter 6. As none of these methods provided a satisfactory prediction of the observed shear connection behaviour nor took into account the beneficial effects of the reinforcing components, a new design proposal is introduced which classifies the shear connection behaviour as ductile or brittle. For common types of shear connections using the Australian types of steel decking, stud strength reduction factors are also given for a simplified strength prediction. In Chapter 8, appropriate design resistance factors are derived to apply the new proposal and the simplified method to the design provisions given in AS2327.1 (Standards Australia 2003a).

The findings of this study and recommendations for further investigations are given in Chapter 9. More detailed information about the various push-out and beam test series can be found in the accompanying test reports published separately.
2 BACKGROUND

2.1 Development of shear connectors in composite beams

The application of steel-concrete composite beams dates back a long time. Initially engineers embedded steel sections into the concrete to increase its strength. A Viennese engineer named Josef Melan introduced a patent for bending steel I-beams to the curvature of an arch and then casting them into concrete. Using his patent numerous bridges were built in Europe and the U.S. As early as 1875, buildings incorporating beams with light steel I-sections surrounded by a ‘box of concrete’ were erected in the U.S. (Huen 1975). With the introduction of composite beams, the use of mechanical shear connectors rose very quickly. In 1903, Julius Kahn received a U.S. patent on “composite structural members” where a metallic member, embedded in another “plastic body of material such as concrete”, should ensure the bond between these two materials.

The demand for composite construction increased in Europe in the 1930s and early 1940s when steel became an expensive construction material. It was then that research on mechanical shear connectors began (Albrecht 1945; Maier-Leibnitz 1941; Ros 1934). Various types of mechanical shear connectors, such as spiral connectors, inclined round steel sections with hooks, bent through 45°, L- and ½ I-sections and channels were tested during that period (Figure 2-1). Generally the connectors were manually welded on the top of the flange of a steel section. It was believed that the shear connection had to be very stiff to ensure satisfactory composite behaviour and prevent slip at the steel-concrete interface.
Figure 2-1: Some of the early shear connectors as illustrated in Klingenberg (1950)

After the Second World War, the established shear connectors as well as some new connectors such as steel straps and stirrups were commonly used for bridge- and building-structures. But things changed dramatically when welded headed studs were introduced in the late 1950s in Great Britain and the U.S. Their application reduced the time consuming and complicated manual welding process of the shear connectors to the steel flange by replacing it with electric arc stud welding which uses a special fast-welding gun (Laurie 1961) and accelerates the process to a few seconds per stud. Hence, the new shear connectors made the application of composite structures much cheaper and more widely spread. Early research on both push-out specimens and composite beams concluded that a uniform spacing of the shear connectors along the length of the beam is satisfactory for composite beams supporting a uniform load or a single load at mid-span (Chapman and Balakrishnan 1964; Slutter and Driscoll 1965). It was further shown that the ultimate capacity of shear connectors rather than their lower ‘critical’ capacity, which had been used for earlier design, provided a good basis for determining the strength and that the performance of composite beams incorporating flexible connectors such as welded studs was similar to the ones using rigid connectors which, at that time, were still preferred in continental Europe. In consequence, headed studs became and still are the dominant method of shear connection.
2.2 Headed stud shear connectors in solid slabs

2.2.1 Load-bearing behaviour

One of the most illustrated models to explain the load transfer of stud connectors in solid slab applications is given by Lungershausen (1988) where four different components which contribute to the total capacity of the connector are defined (Figure 2-2). Initially, the majority of the longitudinal shear force is transferred at the base of the stud into the surrounding concrete (A) where a significant amount of it reacts directly at the weld collar. The multi-axial high bearing stresses in the concrete eventually lead to local crushing failure of the concrete at the bottom of the stud and to a redistribution of the shear forces in areas higher up the shank of the stud (B). Since the top of the stud is embedded in undamaged concrete and cannot deform while the base of the connector is free to move laterally, bending and tensile stresses are induced into the shank of the stud (C). To balance these tensile stresses, compressive forces develop in the concrete under the head of the stud and are thought to activate additional frictional forces (D) at the steel concrete interface. Eventually the shear connection fails when the shank of the stud experiences a combined shear-tension failure right above the weld collar.

![Figure 2-2: Load transfer of a headed stud connector in a solid slab in accordance with Lungershausen (1988)](image.png)

However, the model was neither translated into a design approach nor were the various components, as shown in Figure 2-2, quantified. In particular, the existence...
of the friction component may be questionable. Scheele (1991) argued that the concrete slab may not be in contact with the steel interface, as it was observed in tests that the concrete surrounding the studs separated from the steel section. The solid slab tests described in Chapter 4 also confirmed this observation.

Nevertheless, the model is suitable to describe the main factors influencing the stud behaviour. The area of the stud shank and its ultimate tensile strength obviously influence the shear strength of the stud directly. So does the ductility of the stud material influence the ductility of the shear connection if stud shearing failure is experienced as shown by tests on hot forged and cold-formed studs (Hawkins 1973) or on studs made of high-strength steel (Lyons et al. 1994). The compressive strength and stiffness of the concrete surrounding the studs also influence the behaviour as these properties define the bending and tensile effects induced into the stud shank, hence the height over which the shear forces are transferred into the surrounding concrete. König and Faust (2000) gave a graphic example of this influence when comparing the deformation of headed studs embedded in light-weight concrete of different strengths (Figure 2-3). For normal-weight and normal-strength concrete, it was found from test results that a constant bearing pressure applied over an effective stud height which can be assumed to equal 1.8 times its diameter provided a good approximation (Oehlers 1989). This seems to agree reasonably well with the load transfer of headed studs in fastener applications loaded in shear where the effective height was approximated to be 2 times the stud diameter (Fuchs 1992). The concrete properties do generally influence the stiffness and deformation capacity of the shear connection (see Figure 2-3) but may not necessarily influence its strength. If, for example, a shear connection strength and stiffness is such that the concrete crushing only appears very locally, the bending effect remains limited and so do the tensile forces in the shear connectors. It was shown that at low tensile force in the stud shank, its shear capacity is hardly influenced and the stud effectively fails in pure shear (Bode and Hanenkamp 1985; Johnson et al. 1969). Any further increase in concrete strength cannot result in an increase in the connection strength. The existence of an upper bound of the shear connection strength, which is unrelated to the concrete properties, was confirmed by numerous researchers, e.g. Chapman (1964); Lungershausen (1988); Ollgaard et al. (1971); Roik and Hanswille (1983); Roik et al. (1988).
The influence that the weld of the stud has on the shear connector strength was studied in more detail by Johnson and Oehlers (1981). Numerical investigations showed that the majority of the shear forces are transferred via the weld and that the existence of the weld reduces the bending moment and tensile forces in the shank of the stud significantly. At the same time, a larger weld increases the bearing surface at the concrete-stud interface and reduces the stresses for a given force. The size of the weld has a significant impact on the stud strength and the optimal weld height was found to be 28% to 35% of the stud diameter. An example is given where the stud strength increased by 22% for a shear connector with a 5mm high weld compared to one with no weld. Scheele (1991) reported two push-out tests where one specimen had the weld collar removed from the 19mm diameter studs. The absence of the weld collar resulted in a shearing failure of the studs just above the steel section and a reduction of 25% in shear connection strength and of 1mm in slip capacity. Lungershausen (1988) compared the stud strength of tests which did not have a distinct weld collar with results of corresponding tests. Again, for 19mm diameter studs, the capacity was reduced by about 20-25%. It can be concluded that the existence of the weld collar seems to be one reason for the much higher shear strength of stud connectors which is about 95% of its tensile capacity when compared to the theoretical Huber-von-Mises shear strength which equals 58% of its tensile capacity. The variation in welding collar dimensions may also be a factor.
contributing to the comparatively large scatter in test results across different researchers.

The tensile stresses which develop in the shank of the stud (C in Figure 2-2) can lead to a premature pull-out of the stud surrounded by a cone of concrete from the remaining concrete slab as observed in composite beam tests by Chapman (1964) where 50mm long diameter 19mm studs experienced this particular failure. Longer shear connectors can easily suppress this failure mode. It was found that an embedment depth of four times the stud diameter was sufficient to prevent such a failure (Slutter and Driscoll 1965; Viest 1956). Tensile stresses, where significant due to beam loading arrangements (e.g. bottom flange loading), can reduce the shear stress fracture capacity of the stud. In this case, the shear strength should be based on a tension-shear interaction similar to that used for bolts in steel codes, e.g. AS4100 (Standards Australia 1998b).

The concentrated loads which are initially transferred into the concrete slab at the base of the shear connectors need to be distributed over the effective width of the slab section. In order to compensate for the diagonal compressive concrete struts, transverse tensile stresses develop in front of the shear connector which, if they exceed the tensile capacity of the concrete, can lead to a longitudinal concrete splitting of the concrete slab. Longitudinal shear reinforcement which is usually provided in the base of the concrete slab might accommodate these forces. It was found that the application of transverse reinforcement does not prevent longitudinal splitting failure itself but rather controlled the width and propagation of the longitudinal crack (Johnson and Oehlers 1981; Oehlers 1989). However, where a longitudinal crack appeared in the triaxial compressive zone restraining the shear connector, this zone was significantly weakened and the strength of the shear connection reduced (Oehlers and Park 1992). Longitudinally splitting failure was investigated in great detail by Johnson, Oehlers and Park (Johnson and Oehlers 1981; Johnson and Oehlers 1982; Oehlers 1989; Oehlers and Johnson 1981; Oehlers and Park 1992; Oehlers and Park 1994). The local splitting strength was found to be a function of an effective slab width and height, the concrete tensile strength and the diameter of a stud. In practical applications, longitudinal splitting failure is mainly restricted to narrow concrete slabs, such as edge beams (Johnson and Oehlers 1982).
or beams incorporating concrete haunches (see Chapter 2.2.3), or to beams where the stud connectors are spaced so closely in the longitudinal direction that the lateral tensile stress zones of individual connectors overlap, hence the splitting strength is significantly reduced (Androutsos and Hosain 1992; Davies 1969; Gnanasambandam and Hosain 1995). Tests have established that a longitudinal stud spacing of about 5 time the stud diameter is sufficient to prevent longitudinal splitting for normal strength concrete in internal beam applications (Androutsos and Hosain 1992; Gnanasambandam and Hosain 1995).

### 2.2.2 Design approaches

The design approach that is predominantly used around the world to determine the shear connection strength in static applications is based on the research work performed by Ollgaard et al. (1971). In this method the individual shear connector strength $f_{vs}$ is the lesser of

$$f_{vs} = c_1 \sqrt{f'_c E_c A_{sc}} \quad \text{and}$$

$$f_{vs} = c_2 f_{uc} A_{sc}$$

(2-1)  

(2-2)

where: $c_1$ and $c_2$ are calibration factors based on the analysis of test results; $f'_c$ is the concrete compressive strength; $E_c$ is the concrete elastic modulus; $A_{sc}$ is the cross-sectional area of the shank of the stud; and $f_{uc}$ is the ultimate tensile strength of the shear connector material. The different standards and design specifications provide different values for the factors $c_1$ and $c_2$ to determine the nominal shear connector strength as shown in Table 2-1.

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<tr>
<td>$c_1$</td>
<td>0.39</td>
<td>0.37</td>
<td>0.32</td>
<td>0.32</td>
<td>0.50</td>
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<tr>
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The scatter in the factors can be attributed to the different design and reliability concepts of each particular country as they were largely based on the same test.
Background

results. The values given in ANS/AISC 360-05 (AISC 2005), for example, represent more the mean stud strength while the Australian and European factors are based on a characteristic strength usually resembling a lower 5% fractile. Additionally, each country tends to base their design rules more on the assessments and recommendations of their own researchers.

Based on the evaluation of 110 push tests, a slightly different approach to determine the stud strength was derived by Oehlers and Johnson (1987). The strength of a single shear connector in a composite beam was found to best correlate to:

\[
 f_{vs} = 4.1 \left( \frac{f_{cu}}{f_{uc}} \right)^{0.35} \left( \frac{E_c}{E_{sc}} \right)^{0.40} A_{sc} f_u
\]

(2-3)

where \( f_{cu} \) is the characteristic cube strength of concrete and \( E_{sc} \) is the elastic modulus of the shear connector material. This approach formed the basis for the stud strengths given in BS5950.3 (BSI 1990). In comparison to the previous approach, this model represents the interaction between the steel and concrete properties better, but it does not provide an upper bound for the shear connector strength. Generally both approaches were largely based on the same test results and therefore do provide similar strengths for the common shear connection dimensions and properties.

The various standards also provide tight limits for the application of their particular stud strength approach (Table 2-2). Most of these limits are similar in the different specifications. The purpose of the majority of the limits is to prevent the occurrence of premature concrete-related failure modes such as concrete splitting or stud pull-out, hence to ensure that the expected failure mode is stud shearing and the connection behaves in a sufficiently ductile manner.
Table 2-2: Limits on stud shear connection dimensions in different design specifications

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<tr>
<td>Diameter stud</td>
<td>$d_{bs}$</td>
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<td>16/19</td>
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<td>16-25</td>
<td>13-25</td>
<td>$\leq 19$</td>
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<td>$3d_{bs}$</td>
<td>$3d_{bs}$</td>
<td>$60-95$</td>
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<td>2</td>
<td>2</td>
<td>3</td>
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</tr>
<tr>
<td>Max stud tensile strength</td>
<td>$f_{uc}$</td>
<td>MPa</td>
<td>500</td>
<td>500</td>
<td>450</td>
<td>450</td>
<td>415</td>
</tr>
<tr>
<td>Min long. stud spacing</td>
<td>$s_c$</td>
<td>mm</td>
<td>$5d_{bs}$</td>
<td>$5d_{bs}$</td>
<td>$5d_{bs}$</td>
<td>$5d_{bs}$</td>
<td>$6d_{bs}$</td>
</tr>
<tr>
<td>Max long. stud spacing</td>
<td>$s_c$</td>
<td>mm</td>
<td>$4D_c$, 600</td>
<td>$6D_c$, 800</td>
<td>$6D_c$, 800</td>
<td>$4D_c$, 600</td>
<td>$8D_c$, 915</td>
</tr>
<tr>
<td>Min transv. Stud spacing</td>
<td>$s_x$</td>
<td>mm</td>
<td>$1.5d_{bs}$</td>
<td>$2.5d_{bs}$</td>
<td>$2.5d_{bs}$</td>
<td>$4d_{bs}$</td>
<td>$4d_{bs}$</td>
</tr>
<tr>
<td>Min distance stud - edge of steel flange</td>
<td>$e_D$</td>
<td>mm</td>
<td>20</td>
<td>20</td>
<td>20</td>
<td>20</td>
<td></td>
</tr>
<tr>
<td>Min distance bottom reo – head of stud</td>
<td>mm</td>
<td>30 clear</td>
<td>30 clear</td>
<td>30 clear</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Concrete strength</td>
<td>$f'_c$</td>
<td>MPa</td>
<td>$\leq 40$</td>
<td>$\geq 20$</td>
<td>$\leq 60$</td>
<td>$\geq 20$</td>
<td>$\geq 20$</td>
</tr>
<tr>
<td>Min distance stud - edge of concrete slab</td>
<td>mm</td>
<td>150 clear</td>
<td>$6d_{bs}$</td>
<td>$6d_{bs}$</td>
<td>$6d_{bs}$</td>
<td>$6d_{bs}$</td>
<td></td>
</tr>
</tbody>
</table>

1: with reduction  
2: 60 for $d_{bs}=16$, 75 for $d_{bs}=16,19$ and 95 for $d_{bs}\geq 22$  
3: clear distance between head of studs  
4: measured from edge of shear connector  
5: distance to underside head of stud  
6: including loop reinforcement

2.2.3 Haunches in solid slab construction

Research on composite beams with concrete haunches has been conducted since the early 1930s (Albrecht 1945). Haunches were commonly used in both bridge and building construction to increase the overall depth and stiffness of a beam. Design details also began to appear in codes, e.g. CP117, Part 2 (BSI 1967), and further provisions were then included in subsequent codes. Using the typical haunch in Figure 2-4, the requirements of various codes are detailed in Table 2-3. Typically,
the haunch angle $\theta$ must be less than 45° and stirrup-type transverse haunch reinforcement $A_{sh}$ needs to be included. The haunch reinforcement is taken into account in longitudinal shear resistance calculations for shear planes crossing this reinforcement. The shear connector strengths of studs in concrete haunches are taken as that for solid slab applications although Johnson (1972) recommended using only 80% of this value. The recommendation was based on a study of the beam and push-off tests carried out by Taylor et al. (1970). It is interesting to note that the latest version of Eurocode 4 (CEN 2004) now requires the surface of the connector that resists separation forces (e.g. the underside of the stud connector head) to extend not less than 40mm clear above the bottom haunch reinforcement $a_h$ whereas previously there was no requirement (CEN 1992). In contrast to the earlier version of AS2327.1 (SAA 1980), haunches are no longer included in the current version (Standards Australia 2003a).

![Figure 2-4: Details of a haunch in a solid slab](image)

**Table 2-3:** Comparison of code requirements for haunches

<table>
<thead>
<tr>
<th>Code</th>
<th>$\theta$</th>
<th>$A_{sh}$</th>
<th>$h_{ch}$ (mm)</th>
<th>$e_v$ (mm)</th>
<th>$e_d$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CP 117.2 (BSI 1967)</td>
<td>$b_l &gt; 1.5b_h$</td>
<td>Included</td>
<td>$\geq 50$</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>AS2327.1 (SAA 1980)</td>
<td>$\leq 45^\circ$</td>
<td>Included</td>
<td>$\geq 20$$^1$</td>
<td>$\geq 75$</td>
<td>$\geq 25$</td>
</tr>
<tr>
<td>BS 5950.3 (BSI 1990)</td>
<td>$\leq 45^\circ$</td>
<td>Included</td>
<td>-</td>
<td>$\geq 50$</td>
<td>$\geq 20$</td>
</tr>
<tr>
<td>Eurocode 4 (CEN 1992)</td>
<td>$\leq 45^\circ$</td>
<td>Included</td>
<td>-</td>
<td>$\geq 50$</td>
<td>$\geq 20$</td>
</tr>
<tr>
<td>AS 2327.1 (SA 2003a)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Eurocode 4 (CEN 2004)</td>
<td>$\leq 45^\circ$</td>
<td>Included</td>
<td>$\geq 40$$^1$</td>
<td>$\geq 50$</td>
<td>$\geq 20$</td>
</tr>
</tbody>
</table>

$^1$: to underside of head of the stud connector

The amount of transverse reinforcement to provide sufficient longitudinal shear resistance was derived by Johnson (1970) from ultimate strength methods based on

---
the results of composite beam tests and reinforced concrete tests. However, there
appears to be no guidance in the codes regarding the spacing of the transverse
haunch reinforcement. To account for longitudinal splitting, Johnson (1972)
recommended that the transverse bars should be placed close to the connectors at a
spacing not greater than four times the difference between the side cover to the stud
connectors and the side cover to the transverse bars.

If the local longitudinal splitting strength of the shear connection, which can be
derived in accordance with Oehlers (1989) and Oehlers and Park (1994), is exceeded
which would be the case for most haunch applications, an alternative design concept
derived by Oehlers and Park (1992), even though not specifically aimed at haunched
solid slabs, could also be applied. The maximum shear connector strength of a
longitudinal cracked concrete slab $f_{vs,cr}$ was derived from regression analysis to give:

$$f_{vs,cr} = 0.6f_{vs} + \left( \frac{3A_{sh}s_c}{h_{ec}^2} \right) f_{vs} \leq 0.9f_{vs}$$ \hspace{1cm} (2-4)

where $s_c$ is the longitudinal stud spacing and $h_{ec}$ the effective stud height over which
the shear forces are transferred into the surrounding concrete and which was set to

$$h_{ec} = 1.8d_{bs}. \hspace{1cm} (2-5)$$

It was found that a minimum reinforcement of

$$A_{sh} \geq \frac{0.02h_{ec}^2}{s_c} \hspace{1cm} (2-6)$$

should be provided with a maximum reinforcement bar diameter $d_r$ of

$$d_r \geq 0.4h_{ec} \hspace{1cm} (2-7)$$

In addition, the transverse haunch reinforcement still needs to fulfil the vertical shear
plane reinforcing requirements.

### 2.2.4 Connection enhancement devices

Early research (Chapman and Balakrishnan 1964) already suggested that the
performance of a stud shear connection might be improved by either reducing the
bearing pressure on the concrete at the base of the stud, or by increasing the local
bearing capacity of the concrete. Measures suggested were the use of a threaded stud with a recessed nut turned down to its base or the use of local reinforcement to contain the concrete in the vicinity of the stud.

![Figure 2-5: Helix reinforcement by Sattler (1962)](image1)

![Figure 2-6: Stud embedded in UHPC cover (Hegger et al. 2005)](image2)

The latter suggestion had already been put into practice in Germany (Sattler 1962) where a helix-shaped reinforcing element consisting of a soft 5mm wire with a 50mm diameter was placed around the base of the stud (Figure 2-5). The use of the helix element led to significant higher ultimate capacities and a stiffer overall behaviour of the shear connection. Later research revealed that it was not necessary to hold the helix element in position, hence it could be freely located around the stud without any clipping device (Roik and Lindner 1972). The helix elements were referred to in the previous German design provisions DIN-Richtlinien (DIN 1981) where the shear strength of a stud in a solid slab under working loads could be increased by about 15% if corresponding tests showed that the space between the helix element and stud could be completely filled with concrete on site. However, the helix element appeared to have not been generally accepted as it was not included in the latest version of the German standard, DIN V 18800-5 (DIN 2004). Strictly speaking, the general allowance to increase the shear connection strength where this element is applied was not justified as the element can only have an impact on the shear connection strength if the concrete properties have an impact on the connection behaviour which, according to Equations (2-1) and (2-2), is only the case for lower concrete strengths or lightweight concrete. However, as the DIN-Richtlinien (DIN
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1981) generally provided lower shear connector strengths (see Table 2-1), there are no safety concerns for existing structures including this helix element.

Research by Hegger et al. (2005) cast the base of the shear connector in a 50x50mm ring of ultra-high performance concrete (UHPC), which is characterized by a concrete strength of more than 180MPa and a tensile splitting strength of 23MPa, before the remaining slab was poured (Figure 2-6). Where this construction was used in normal strength concrete, a significant improvement in stud strength was achieved since any bending effects at the base of the shear connector were suppressed. But again, a load higher than the strength of the stud in pure shear cannot be achieved. Consequently, the beneficial effect on strength can only be observed up to a certain concrete strength and stiffness. As would be expected, the use of this construction in high strength concrete did not lead to an increase in strength, but it provided a significant increase in ductility which is usually a major concern for shear connections in high-strength concrete. If an economical application of the UHPC cover can be found remains to be seen.

2.3 Headed stud shear connection in composite slabs

2.3.1 Steel decking in composite structures

Profiled steel decking was first used in the construction of concrete slabs in building structures in the U.S. in the late 1950s. Initially, the corrugated steel deck was nothing more than a cheap permanent formwork for the concrete slab which additionally provided ducts for electrical and other conduits. But soon embossments and depressions were added to the steel decking which ensured interlock with the concrete slab and made it an integral part of the floor construction. By the end of the 1960s, composite slabs were the most common floor system used in high-rise buildings in the U.S. with the Sears Tower in Chicago just one of its famous applications. The next logical step was to use the composite slab as a structural component of a composite beam construction. In the beginning, holes were cut into the deck in order to weld the stud shear connectors onto the flange of the steel
section, a technique still carried out in some countries today, e.g. Germany, but soon it became possible to weld the connectors directly through the sheeting if its diameter did not exceed \(\frac{3}{4}\) in. (19mm). Today, there is a large number of different types of steel decking available worldwide; many of them still resemble their American predecessors with heights of either 2in. or 3in. (51mm / 76mm) and a panel width of either 1ft or 2ft (300mm / 600mm). Table 2-4 shows the types of decking which are currently available in Australia:

**Table 2-4: Steel deckings currently available in Australia**

<table>
<thead>
<tr>
<th>No</th>
<th>Manufacturer</th>
<th>Decking</th>
<th>Geometry</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Fielders Australia</td>
<td>KF57®</td>
<td><img src="image1" alt="Diagram" /></td>
</tr>
<tr>
<td>2</td>
<td>Stramit</td>
<td>Condek HP®</td>
<td><img src="image2" alt="Diagram" /></td>
</tr>
<tr>
<td>3</td>
<td>Fielders Australia</td>
<td>RF55®</td>
<td><img src="image3" alt="Diagram" /></td>
</tr>
<tr>
<td>4</td>
<td>Lysaght</td>
<td>Bondek®</td>
<td><img src="image4" alt="Diagram" /></td>
</tr>
<tr>
<td>5</td>
<td>Fielders Australia</td>
<td>KF70®</td>
<td><img src="image5" alt="Diagram" /></td>
</tr>
</tbody>
</table>
The different types of steel decking geometries can be divided into several main groups: closed-rib profiles (No. 1, 2 and 7), narrow open or dovetailed-rib profiles (No. 3 and 4) and wide-open rib profiles (No. 5, 6 and 8). Geometry No 8 is a very high profile which is usually laid on the bottom flange of a steel beam and does not interfere with the shear connection of the beam. Therefore this geometry is outside the scope of this thesis.

The steel decking is usually laid across several secondary beams of a steel structure. Hence, it runs transverse to the steel beam and creates voids in the direction of the longitudinal shear which can interrupt the flow of force. In primary beam applications, the decking runs parallel to the beam effectively creating a concrete haunch in the longitudinal direction. In the following, the applications of steel decking in secondary and primary beams are treated separately as their weakening effects on the shear connection differ.
2.3.2 Secondary beam applications

2.3.2.1 Early research

Early research (Fisher 1970; Robinson 1967; Robinson 1969) observed that the mode of deformation and failure of a secondary beam type of shear connection differed from that of shear connection in solid slab applications and that the shear connectors could typically not reach their full capacity. Some experienced less than 30% of the strength they would have achieved in a solid slab. The predominant failure mode observed was a horizontal cracking of the concrete ribs which, in the further course of this thesis, is referred to as ‘rib shearing’ failure. It was found that the geometry of the concrete rib had a big influence on the overall performance of the shear connection and the sheeting pans needed to be formed wide enough to ensure adequate cover of the studs. It was also noted that with increasing width of the concrete slab the shear connection strength increased but not to the level a similar connection would reach in a solid slab application, even if very wide specimens were tested. This led to the conclusion that cracks similar to rib shearing cracks still appeared in the vicinity of the shear connection but did not propagate across the full width of the concrete slab. Based on an empirical analysis of previous composite beam tests, a design suggestion, which determined the composite slab shear connection strength by multiplying its solid slab strength with a reduction factor consisting of the width to height ratio of the concrete rib, was provided (Fisher 1970).

Horizontal push-off tests on 1800mm wide specimens by Iyengar and Zils (1973) experienced a “shear cone failure above the studs” where the stud connectors, surrounded by a cone of concrete, separated from the solid part of the cover slab. The same failure mode was identified by later research (Hawkins and Mitchell 1984; Lloyd and Wright 1990) as ‘stud pull-out’ failure. It was observed that the stud strength increased with increasing embedment depth of the studs into the solid cover part of the concrete slab as the size of the concrete failure cone also increased. If the number of studs per concrete rib was increased, a significant reduction in the individual stud strength was observed and the individual concrete failure cones
surrounding the studs overlapped. In some of the tests, the studs were placed off-centre in the concrete ribs and it was observed that, if placed on the side which provided the larger amount of concrete cover at the stud bearing zone, the stud strength increased. Push-out and composite beam tests performed by Robinson and Wallace (1973) also supported the findings of a decrease in connector strength with lower embedment depth or a larger number of studs per concrete rib. In this particular research, the predominant failure mode was the base of the studs punching through the ribs of the steel decking, i.e. ‘rib punch-through’ failure.

### 2.3.2.2 Reduction factor approaches

An extensive program of composite beam tests undertaken by Grant et al. (1977) formed the basis for the most common reduction factor approach. After performing 17 full-scale beam tests and evaluating 58 beam tests done previously, the following reduction factor $k_i$ of a solid slab shear connection was suggested for secondary composite beam applications:

$$k_i = \frac{0.85 \, b_{cr} \left( \frac{h_r - h_c}{h_r} \right)}{\sqrt{n_x \, h_r}} \leq 1.0$$

(2-8)

where $n_x$ is the number of headed studs per rib, $h_r$ the rib height, $b_{cr}$ the mid-height width of the pan and $h_c$-$h_r$ the embedment depth of the stud into the covering solid part of the slab. Over the years, this design provision has proved to be extremely popular and was, or is still used, in the majority of the standards to derive the capacity of secondary beam shear connectors. However, some aspects of the research work of Grant et al. (1977) require clarification:

- The reduction factor was derived in a pure empirical manner from testing of steel decking geometries with wide-open ribs having a width between 1½in. to 3in. (38mm-76mm) with a width to height ratio of 1.5 and 2.0. All of the studs in the tests were placed in the central position of the ribs and embedded 38mm deep in the concrete cover slab. Strictly speaking, empirical approaches should only be valid for the boundary conditions that they were derived for and not be extrapolated to other dimensions.
The beam tests considered showed various modes of failure such as rib-shearing, stud pull-out or rib punch-through (dominant failure mode). These different failure modes are only covered by a single equation. Hence, the scatter of the test results is very large.

The failure modes experienced in the composite slab application are different from solid slab failures but the strengths are still based on the factors influencing stud-shearing failure, e.g. the stud tensile strength would not have an effect on concrete-related failure modes but is still used in the determination of the shear connector strength.

The stud strength in solid slab applications was determined using the results of push-out tests where the connector force can be determined directly. In this study, the shear connection strengths for secondary beam applications was determined indirectly by comparing the ultimate bending moment capacity of composite beam tests with their theoretical capacity if solid slab shear connections would have been used. Thereby, the position of the resultant of the concrete compressive force was estimated which inevitably leads to inaccuracies. It should be noted that small changes in the moment capacity result in comparatively large changes in the shear connector force (as shown in Chapter 2.5).

Without strain measurements in the steel beam section, no statement about the deformation capacity of an individual connector can be made (see also Chapter 2.5).

Consequently, the design approach suggested in Grant et al. (1977) has been the cause of much controversy. Studies of various researchers (Lawson 1992; Roik et al. 1988; Stark and van Hove 1991) compared the reduction factor approach with numerous push-out tests and found that the use of Equation (2-8) resulted in a high scatter of the tests results and might yield unconservative predictions in particular where pairs of studs were used or where the concrete rib was relatively wide. It was also criticised that Equation (2-8) does not provide any guidance for studs placed off-centre in a concrete rib (Lawson 1992). An application of the reduction factor was not recommended in any of these studies. However, based on a statistical analysis, it was proposed that reducing the factor of 0.85 to 0.70 in Equation (2-8) would provide a conservative estimate (Stark and van Hove 1991). Another analysis of the
application of this reduced factor found that the strength of the stud connection could still be overestimated, in particular where thin or pre-holed steel decking was used (Hanswille 1993). Upper limits $k_{t,max}$ depending on the sheeting thickness, the number of shear connectors and the pre-holing of the steel decking were introduced to accommodate the safety concerns. These limits were adopted in Eurocode 4 (CEN 2004) and are shown in Table 2-5. In contrast, BS5950.3 (BSI 1990) still applies the unaltered form of Equation (2-8) with the exceptions that the strength of stud pairs is restricted to an upper limit of 0.8 and that for studs placed off-centre in a weak position the effective width of the concrete rib is reduced to $2e$ where $e$ is the distance between stud and steel decking. A statistical analysis of 183 secondary beam push-out tests found that the Eurocode 4 method considering the limit $k_{t,max}$ provides a large scatter and unsafe results in many cases where several tests only reached 70% of their predicted strength (Johnson and Yuan 1998a). In the same work, it was concluded that a more sophisticated approach which distinguishes between the various failure modes was needed.

Table 2-5: Upper limits $k_{t,max}$ according to Eurocode 4 (CEN 2004)

<table>
<thead>
<tr>
<th>Number of stud connectors per rib</th>
<th>Thickness $t$ of sheet (mm)</th>
<th>Studs not exceeding 20mm in diameter and welded through profiled steel sheeting</th>
<th>Profiled sheeting with holes and studs 19mm or 22mm in diameter</th>
</tr>
</thead>
<tbody>
<tr>
<td>$n_r = 1$</td>
<td>$\leq 1.0$</td>
<td>0.85</td>
<td>0.75</td>
</tr>
<tr>
<td></td>
<td>$&gt; 1.0$</td>
<td>1.00</td>
<td>0.75</td>
</tr>
<tr>
<td>$n_r = 2$</td>
<td>$\leq 1.0$</td>
<td>0.70</td>
<td>0.60</td>
</tr>
<tr>
<td></td>
<td>$&gt; 1.0$</td>
<td>0.80</td>
<td>0.60</td>
</tr>
</tbody>
</table>

Lawson (1992) modified the reduction factor given by Equation (2-8) and differentiated between two separate cases depending on the rib width:

$$ k_i = \frac{0.75 b_{cr}}{\sqrt{n_x}} \frac{h_t}{h_t + h_e} \leq 1.0 \quad \text{for } b_{cr} \leq 2 h_t \quad (2-9) $$

$$ k_i = \frac{1.5}{\sqrt{n_x}} \frac{h_t}{h_t \left( \frac{2h_t}{b_{cr}} \right) + h_e} \leq 1.0 \quad \text{for } b_{cr} \geq 2 h_t \quad (2-10) $$

However, the statistical analysis performed by (Johnson and Yuan 1998a) showed similar types of scatter and safety concerns for this approach.
Another approach, also based on the reduction of the solid slab strength, was recently introduced by Rambo-Roddenberry (2002). It was derived on the basis of the evaluation of 202 push-out tests performed at the Virginia Polytechnic Institute in the U.S. The approach was slightly modified and incorporated into the latest version of the U.S design provisions ANSI/ASIC 360-05 (AISC 2005). There, the stud strength can be determined in accordance with:

\[
 f_{sv} = 0.5A_{sc} \sqrt{f'\text{c}\text{,}}\frac{E_{c}}{R_{g}R_{p}f_{sc}A_{sc}} \leq R_{g}R_{p}f_{sc}A_{sc} \tag{2-11}
\]

where

- \( R_{g} = 1.00 \) for single studs
- \( R_{g} = 0.85 \) for stud pairs
- \( R_{g} = 0.70 \) for three or more studs
- \( R_{p} = 0.75 \) for \( e \geq 51\text{mm} \) (strong position studs)
- \( R_{p} = 0.60 \) for \( e \leq 51\text{mm} \) (weak position studs)

\( e \) is the distance from edge of stud shank to mid-height of steel deck in load bearing direction of the stud.

This approach is evaluated in detail in Chapter 6.2.1.3.

### 2.3.2.3 Failure modes

The research of Hawkins and Mitchell (1984) was the first to differentiate between the four failure modes observed in secondary beam push-out tests: Stud-shearing failure, stud pull-out failure, rib shearing failure and rib punch-through failure. As these four failure modes were also experienced in the tests described in Chapter 4, their behaviour is described in detail in Chapter 4.4. In the following, the various strength prediction methods available to describe the particular failure modes are presented:

**Stud shearing**

As similar stud shearing behaviour is also observed in solid slab applications, the strength can be determined accordingly, hence Equations (2-1) and (2-2) can be applied (Hawkins and Mitchell 1984; Johnson and Yuan 1998b).
**Stud pull-out**

Hawkins and Mitchell (1984) observed that stud pull-out failure occurred very suddenly at lower loads and provided significant less ductility compared to stud shearing failure. It was thought that the failure was triggered by high tensile forces acting in the shank of the stud resulting in a pull-out of the connector surrounded by a cone of concrete similar to fastener applications loaded in tension. A design concept was developed which assumed a cone-shaped failure surface propagating vertically into the concrete slab from the underside of the head of the stud at an angle of 45° (Figure 2-7). The stud capacity against stud pull-out failure $P_{SP}$ was determined to be

$$P_{SP} = k \ A_{cone} \sqrt{f'_{c}} . \tag{2-12}$$

where $A_{cone}$ is the projected horizontal failure cone surface, $(f'_{c})^{0.5}$ a factor representing the influence of the concrete tensile strength ($f'_{c}$ in MPa) and $k=0.49$ an empirically derived constant as the amount of tensile force acting in the stud shank is unknown. Later research on different types of steel decking geometries determined the constant to be $k=0.35$ for a 76mm high decking geometry and to be $k=0.61$ for a 38mm high decking geometry (Jayas and Hosain 1988). Equation (2-12) is still applied in the Canadian design specifications CSA S16-01 (CSA 2001) to determine the shear connection strengths in secondary beam applications.

The work of Lloyd and Wright (1990) investigated the pullout failure surface in more detail and found slightly different dimensions of the failure cone (Figure 2-8). Using the new geometry, the empirical constant was determined to be $k=0.35$, but the large scatter of the test results was criticized and it was found that the following relationship provided a better fit to the test data:

$$P_{SP} = \left( A_{cone} \sqrt{f'_{cu}} \right)^{0.34} \tag{2-13}$$
A more fundamental approach was developed by Bode and Künzel (1988) who performed a series of 40 pull-out tests on headed studs embedded in a composite slab and determined the tensile capacity of a shear connection against stud pull-out $T_{SP}$ to be

$$T_{SP} = 15.6 \ h_{eff}^{1.5} \ \sqrt{f'_{c}} \ \ \ (2-14)$$

where the effective height of the shear connector $h_{eff}$ was taken as the average between its height in the transverse direction (complete stud height) and in the longitudinal direction (stud height above steel decking):

$$h_{eff} = \frac{h_c + (h_c - h_t)}{2} \ \ \ (2-15)$$

This approach is more related to the methods used in fastener applications (Bode and Hanenkamp 1985; Eligehausen et al. 1992) as it takes the size effect of the concrete failure surface into account. However, the amount of tensile force acting in the stud...
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in relation to the applied shear force is not known. A simple strut and tie model was proposed which split the diagonal compressive strut $C$, which develops in front of the stud, into a vertical and horizontal force component acting in the shank of the stud (Figure 2-9). The stud pull-out capacity can then determined to be

$$P_{SP} = T_{SP} \times \frac{1}{\tan \alpha_C}.$$ \hspace{1cm} (2-16)

As $\alpha_C$ is not known, an iterative process has to be employed using a shear-tension interaction for the stud connector strength. This makes the proposal somewhat complex to use.

![Figure 2-9: Strut and tie model according to Bode and Künzel (1991)](image)

$$P_{SP} = T_{SP} \times \frac{1}{\tan \alpha_C}.$$ \hspace{1cm} (2-16)

A similar approach based on a more complex strut and tie model was proposed by Jenisch (2000). Here, the resultant of the horizontal shear force, which was assumed to be transferred into the surrounding concrete at a height of $d_{bs}$ measured from the base of the shear connector, was split into two components; one which is transferred directly into the concrete compressive zone and one which is anchored via the base panel of the sheeting and transferred into the concrete at the rear side of the shear connector (Figure 2-10). The pull-out strength of a stud was derived in similar fashion as in Bode and Künzel (1991), also taking into account the influence of the size effect in concrete, to be

$$T_{SP} = 15.64 h_{eff,l}^{0.75} h_{eff,q}^{0.75} \sqrt{f_{cu}}$$ \hspace{1cm} (2-17)

where $h_{eff,l}$ is the effective height in the longitudinal direction (to the underside of the shear connector) and $h_{eff,q}$ the height to the underside of the shear connector in the transverse direction, i.e. $h_{eff,l} = h_c - h_{head} - h_i$; $h_{eff,q} = h_c - h_{head}$. If the sheeting component
in Figure 2-10 is neglected, the horizontal shear capacity against stud pull-out failure was determined by applying the strut and tie model to Equation (2-16) with $\alpha_C$ amounting to

$$\tan(\alpha_C) = \frac{h_{c1}}{e} = \frac{h_c - d + 0.2(D_c - h_c)}{e}.$$  \hspace{1cm} (2-18)

A completely different approach to determine the concrete pullout strength was proposed by Johnson and Yuan (1998b) which bases the concrete pull-out capacity on the torsional resistance of the concrete rib. From tests, it was observed that the concrete rib rotates around point A as shown in Figure 2-11. It was assumed that the slip $\delta_c$ of the stud causes a rotation $\theta_r$ in the concrete rib and the cover slab can accommodate the resulting vertical uplift $\delta_v$. Hence, the relations between the concrete rib deformations were obtained from geometry to be

$$\theta_r = \frac{\delta_c}{h_r}, \hspace{1cm} \delta_{sv} = \left(\frac{b_{cr}}{h_r}\right)\delta_c, \hspace{1cm} \delta_{ev} = \left(\frac{e}{h_r}\right)\delta_c.$$  \hspace{1cm} (2-19)

It was further assumed that the initial failure of the prism is along the surface ABC followed by a torsional cracking at an unspecified transverse distance from the shear connection. Neglecting the work component of the initial failure prism, the work equation was given as

$$f_{c't'} \delta_r - T \delta_v = W_u$$  \hspace{1cm} (2-20)

where $W_u$ is the internal torsional work component. Using the sand-heap analogy $W_u$ was derived to be

$$W_u = \nu_{tu} H^2 \left(b_{cr} - \frac{H}{3}\right)\theta_r.$$  \hspace{1cm} (2-21)

where $\nu_{tu}$ is the torsional strength of concrete and $b_{cr}$ and $H$ are the dimensions of a fixed end rectangular concrete cross-section subject to tensile torque. It was found that if the height was taken as $H=0.75h_c$ the model related well to test results. From Equations (2-19) to (2-21) the concrete pullout strength can then determined to be

$$P_{sp} = \frac{0.56\nu_{tu} h_c^2 \left(b_{cr} - \frac{h_c}{4}\right)}{h_r} + T_{sp} \frac{e}{h_r}.$$  \hspace{1cm} (2-22)

Similar to the previous approaches, the stud strength is influenced by the tensile force acting in the shank of the shear connector.
Rib shearing failure

In Hawkins and Mitchell (1984), it was found that specimens experiencing rib shearing failures had a very low strength and that this type of failure should be avoided in applications where the shear connection is close to a free transverse concrete edge such as in edge beams. However, no model to describe this failure was developed. According to the later research (Lloyd and Wright 1990), Equation (2-13) can be applied to determine the stud rib shearing capacity \( P_{RS} \) if the size of the projected failure cone is altered to cover the full width of the concrete flange outstand. It was also stated that if a certain breadth of the specimen is exceeded, concrete pullout failure instead of rib shearing failure would occur.

Patrick et al. have highlighted the very brittle behaviour of this failure mode in numerous publications (e.g. Patrick and Bridge (2002a); Patrick et al. (1995)) and have described the beneficial effects of a waveform type of reinforcement which passes the horizontal failure surface in a vertical manner (see also Chapter 2.3.4). Based on the observations that the cracking was always initiated at the top back corner of the concrete rib and propagated horizontally across the rib width, rib shearing was assumed to be a bending related failure mode (in contrast to the
previous model which assumed a pure tensile failure of the stud connector. In a first crude model (Patrick and Bridge 2002a), the rib was simplified as a lever arm restrained over an effective width $b_{eff}$ by the concrete cover slab. It was further assumed that the interface between concrete rib and solid cover part remains plane and that the bending resistance of the concrete rib is exhausted when the stresses at its rear side exceed the concrete tensile strength $f_t$ (Point A in Figure 2-12). This led to the following stud capacity against rib shearing:

$$P_{RS} = \frac{f_t b_{eff} b^{2}_{cr}}{6h_{cr} n_x}.$$  \hspace{1cm} (2-23)

The effective width was set to $b_{eff} \leq 600$mm depending on the outstand of the nearest transverse concrete edge.

![Figure 2-12: Rib shearing approach according to Patrick and Bridge (2002a)](image)

The failure mode of rib shearing is currently also included as a Type 4 longitudinal shear failure in AS2327.1 (Standards Australia 2003a).

**Rib punch-through failure**

In Lungershausen (1988), it was assumed that, after the concrete bearing zone at the base of the stud has failed, shear forces can only be transferred into the compressive zone via bending of the shank of the stud. Hence, the rib punch-through strength is
reached when two plastic moment hinges have developed in the shank of the stud, one above the stud weld, the other at the end of the concrete bearing zone (Figure 2-13). The rib punch-through capacity can then be determined to be

$$ P_{RPT} = 2 \frac{M_{uc}}{a} $$  \hspace{1cm} (2-24)

where $M_{uc}$ is the plastic moment capacity of the circular stud shank:

$$ M_{uc} = f_{uc} \frac{d_{bs}^3}{6} $$  \hspace{1cm} (2-25)

and $a$ is the distance between the two plastic hinges, which is influenced by the geometry of the sheeting and the diameter of the stud shank. It was derived empirically from test data to:

$$ a = \left[ 0.8 \left( \frac{h_s}{b_{cr}} \right)^2 + 0.6 \right] d_{bs} $$  \hspace{1cm} (2-26)

![Figure 2-13: Rib punch-through approach according to Lungershausen (1988)](image)

It was recommended to use this approach rather than Equation (2-8) to determine the capacity of the shear connection in secondary beams since its comparison with the results of known push-out tests provided a more conservative estimate and a smaller scatter (Roik et al. 1988). However, the analysis by Johnson and Yuan (1998a) concluded that this approach does not provide satisfactory agreement with the results of push-out tests and can overestimate the strength of the connection. In the tests described in Chapter 4, it was observed that the failure of the bearing zone at the base
Background

of the stud resulted in an immediate drop in the shear connection capacity, hence it
defined the maximum strength of the connection. At that stage, the shear connection
had experienced low slips and no plastic hinges in the shank of the stud had yet been
observed. Consequently, the plastic stud bending capacity can only provide an
estimate of the strengths which can be experienced at larger slips and not at the low
slips at which the bearing zone failure occurs.

Another approach, based on the strut and tie model shown in Figure 2-10, was
developed by Jenisch (2000). Here, a failure of the concrete strut \( C_1 \) at point A is
assumed to describe rib punch-through failure. The capacity of the compressive strut
is determined to be

\[
C_1 = A_{C1} \nu f_{cu}
\]

(2-27)

where \( A_{C1} \) is the cross-sectional area of the strut which was derived to

\[
A_{C1} = b_{C1,\text{eff}} \frac{d_{bs}}{\cos(\alpha_C)}.
\]

(2-28)

It was assumed that, in both transverse directions, the shear connector load spreads
with an angle of \( 35^\circ \) towards the transverse so that the total effective width of the
concrete strut amounts to

\[
b_{C1,\text{eff}} = 2e \tan 35^\circ \cos(\alpha_C) + s_x
\]

(2-29)

with \( \alpha_C \) to be determined in accordance with Equation (2-18) and where \( s_x \) is the
transverse spacing of the shear connectors if stud pairs are used. The factor \( \nu \) in

Equation (2-27) was determined to be \( \nu = 0.8 \) for concrete slabs subject to
compressive stresses and \( \nu = 0.6 \) for concrete slabs subject to tensile stresses. Once
the capacity of the concrete strut is known, the horizontal shear capacity against rib
punch-through failure can be determined to:

\[
P_{RPT} = C_1 \cos(\alpha_C) + T_{sh}.
\]

(2-30)

where \( T_{sh} \) is the force component transferred by the steel decking in tension. For the
determination of \( T_{sh} \) it is referred to Jenisch (2000). However, for the trapezoidal
type of steel decking geometries investigated in this thesis, it was found that this
component typically accounted for only a small increase in shear connection strength
of between 1kN to 3kN (see Chapter 6.2.2.2), hence it could also be neglected when determining the shear connection strength for these types of decking geometries.

Johnson and Yuan (1998b) developed yet another approach to predict the rib punch-through capacity which also takes a force component transferred by the steel decking into account. The model assumes that the diagonal concrete compressive strut in front of the shear connector is resisted by biaxial concrete compressive strength. Furthermore, the steel decking right behind the shear connection is assumed to yield over an effective width \(b_{esh}\) when failure occurs. In accordance with Figure 2-14, equilibrium in the two directions is obtained to:

\[
P_{RPT} = T_{y,sh} + b_{eff} f'_{c} x, \quad (2-31)
\]

\[
T_c = b_{eff} f'_{c} y \quad (2-32)
\]

where \(b_{eff}\) is the effective width of the concrete compressive zone and \(T_{y,sh}\) the tensile strength of the steel decking which can be determined to be

\[
T_{y,sh} = b_{esh} f'_{ysh}. \quad (2-33)
\]

From geometry, the relationship between \(x\) and \(y\) can be expressed as

\[
x = \frac{(e - y)}{h_r} y \quad (2-34)
\]

where the dimension \(x\) experiences its maximum length of \(x_{max}=e^2/4h_r\) when \(y=e/2\). Substituting these maximum values in Equations (2-31) and (2-32) provided the horizontal rib punch-through strength as a function of the vertical stud force \(T_c\):

\[
P_{RPT} = b_{esh} f'_{ysh} + T_c \frac{e}{2h_r} \quad (2-35)
\]

where the effective width of the steel decking \(b_{esh}\) was derived from test results to be

\[
b_{esh} = 1.8 \left( e + h_e - h_r \right). \quad (2-36)
\]

The method is evaluated in more detail in Chapter 6.2.2.1.
2.3.3 Primary beam applications

When the steel decking is laid parallel to the steel beam, a concrete haunch similar to solid slab applications (as shown in Figure 2-4) is created along its length. The concrete haunch can be narrow using unsplit steel decking or wider where the steel decking is split over the beam. In contrast to solid slab construction (see Chapter 2.2.3), the haunches are typically not reinforced. This configuration has often been neglected in research work and only very limited test data is therefore available to date.

2.3.3.1 Design specifications

Early research indicated that the shear connection was not significantly affected by the geometry of the ribs and behaved in a similar manner to haunched solid slab shear connections (Grant et al. 1977; Hawkins and Mitchell 1984). However, due to the lack of test data, a reduction factor $k_i$ similar to secondary beam applications was introduced:

$$k_i = 0.6 \frac{b_{cr}}{h_r} \left( \frac{h_c}{h_r} - 1 \right) \leq 1.0$$  \hspace{1cm} (2-37)
where $k_i$ is applied to the strength of the connector in a solid slab. This factor is still used in most of the major codes, e.g. Eurocode 4 (CEN 2004) and BS5950.3 (BSI 1990). The factor is mainly influenced by the geometry of the haunch and neglects important parameters such as the stud spacing, numbers of studs at a transverse cross-section and the proximity of the studs to the steel decking.

The current Canadian standard CSA S16-01 (CSA 2001) adopts a similar reduction factor approach which was derived from a regression analysis of numerous push-out and composite beam test results (Androutsos and Hosain 1992; Gnanasambandam 1995; Wu 1998):

$$ k_i = 0.92 \frac{\beta}{h_r} \left( \frac{f_c}{f_y} \right)^{0.8} + 11 s_c d_{bs} \left( \frac{f_c}{f_y} \right)^{0.2} . $$

Compared to Equation (2-37), the approach includes some additional factors such as the stud diameter, the concrete compressive strength and the longitudinal stud spacing $s_c$. The reduction factor is only applied if the average width-to-height ratio of the haunch is less than 1.5.

The reliability of the stud connector strength reduction formula given in Equation (2-37) has been assessed by Johnson and Yuan (Johnson 1999; Johnson 2000; Johnson and Yuan 1998a). The method was found to be unsafe for certain failure modes and “as it did not predict the failure mode, it should not be used without a big penalty factor”. It is interesting to note that in the latest version of Eurocode 4 (CEN 2004) for the case where the sheeting is not continuous across a primary beam and is not appropriately anchored to the beam, the haunch and its reinforcement must satisfy the requirements of a solid slab haunch which requires well-anchored haunch reinforcement located at least 40mm below the underside of the head of the stud connector and vertical stirrup reinforcement (see Chapter 2.2.3). This differentiation suggests that the unsplit decking in a primary beam application can adequately replace the haunch reinforcement which also provides resistance to vertical separation. The low strength separation failures reported in the literature (Gnanasambandam 1995; Wu 1998; Yuan 1996) and the push-out tests described in Chapter 5 would suggest otherwise.
2.3.3.2 Failure modes

The empirical approaches of Equations (2-37) and (2-38) do not differentiate between the various possible failure modes. Apart from stud shearing, the tests reported in the literature (Bridge et al. 2004; Gnanasambandam 1995; Johnson and Yuan 1998a; Wu 1998; Zaki et al. 2003) have identified two general modes of failure that are referred to as a ‘longitudinal splitting’ type and ‘haunch shearing’ type. The failure modes are described in detail in Chapter 5.4. In the following, some of the existing strength provisions for these two failure modes are presented.

Longitudinal splitting failure

As described in Chapter 2.2, longitudinal splitting was initially observed in solid slab beam tests by Davies (1969). It was found that the splitting crack was first induced in the lower half of the slab and then propagated to the upper surface where it became visible. It was recommended to reinforce the bottom side of a slab against splitting. Sufficient transverse bottom reinforcement in combination with appropriate longitudinal stud spacing is now required in all current design provisions to suppress the effects of longitudinal splitting in solid slabs. However, no bottom reinforcement is required in composite slabs with the ribs running parallel to the steel section, hence longitudinal splitting of the haunch is resisted solely by the tensile strength of the concrete or, in the case where the sheeting is unsplit (i.e. continuous), to some degree by the sheeting.

A three-dimensional model for the splitting strength of unreinforced haunches has been developed by Oehlers and Bradford (1995) which is based on a patch load applied to a concrete prism resulting in a stress distributions similar to the ones observed in anchorage zones of post-tensioned concrete members:

\[
P_{LS} = \frac{0.6\pi b_{eh} h_{ec} f_t}{\left(1 - \frac{d_{bc}}{b_{eh}}\right)^2} \frac{0.6\pi b_{eh} h_{ec} f_t}{1 - \left(\frac{h_{ec}}{D_c}\right)^2} \left(\frac{h_{ec}}{D_c}\right)
\]

(2-39)

where \( b_{eh} \) is the effective width of concrete haunch, \( h_{ec} \) the effective shear connector height which can be determined in accordance with Equation (2-5) for normal density concrete, \( D_c \) the overall thickness of the concrete slab and \( f_t \) the concrete
Background

tensile strength which can be approximated to \( f_t \approx 0.5(f'_c)^{0.5} \). The two components of Equation (2-39) reflect the transverse splitting resistance of the haunch and the vertical splitting resistance obtained from the vertical dispersion of the concentrated shear forces into the concrete cover slab. The effect, if any, of the sheeting is ignored. The inclusion of the concrete tensile strength highlights the brittle nature of this type of failure.

The approach given by Equation (2-39) was modified by Johnson and Yuan (1998b). The factor \( D_c \) was replaced by \( 2D_c \) which was found to better correlate to test results as already suggested earlier (Oehlers and Bradford 1995). Furthermore, the effective shear connector height \( h_{ec} \) was replaced by the factor \( h_{es} \) which reflects an effective patch height and was assumed to exceed the full height of the concrete haunch. From empirical evaluation, it was found that with increasing haunch width this excess became smaller, so that \( h_{es} \) was determined to be

\[
h_{es} = \frac{h_r + h_y}{2} \quad \text{for } b_{eh} \leq 1.5h_c
\]

\[
h_{es} = h_r + \left( 2.4 - \frac{b_{eh}}{h_r} \right) \frac{h_r - h_y}{1.8} \quad \text{for } b_{eh} > 1.5h_c
\]

Adopting these changes and rearranging Equation (2-39), the splitting strength for one row of stud connectors located in the centre of a haunch is given as

\[
P_{LS} = 4.8\pi f_t \left[ \frac{b_{eh} h_{es}^3}{8(b_{eh} - d_{bs})^2} + \frac{D_c^3 d_{bs}}{(2D_c - h_{es})^2} \right] \quad (2-42)
\]

Haunch shearing failure

Early research (Grant et al. 1977) already advised “to insure that shearing of the concrete will not occur on a failure plane over the top of the connectors”. This failure plane is classified as a Type 3 shear surface in AS 2327.1 (Standards Australia 2003a) which requires well-anchored transverse reinforcement with its top face located at least 30mm below the top of the shear connectors. However, even this may not be sufficient if the spacing of this reinforcement is too large (Patrick and Bridge 2002a). Therefore, an additional requirement that limits the longitudinal stud spacing, depending on the clear distance between the head of the stud and the bottom
layer of reinforcement, was proposed which is intended to ensure that the transverse reinforcement bars are effective (see Figure 2-15). There is no shear failure surface equivalent to a Type 3 surface defined in many other standards for the case of sheeting ribs running parallel to the supporting beam, e.g. BS5910.3 (BSI 1990) or Eurocode 4 (CEN 2004). Hence, there is no requirement to provide reinforcement across this shear surface.

![Diagram](image)

**Figure 2-15:** Maximum longitudinal spacing of connectors for conventional shear surface failures as proposed by Patrick and Bridge (2002a)

But even if the minimum embedment and longitudinal spacing requirements are fulfilled, concrete cones similar to the ones shown in Figure 2-15 might be able to develop and undermine the transverse reinforcement bars. This behaviour characterizes haunch shearing failure and prevents the activation of transverse reinforcement laid on top of the steel decking altogether. The tests by Androutsos and Hosain (1992), for example, provided a bottom layer of reinforcement although it is not known if the detailing fulfilled the requirements of Figure 2-15, and still experienced a haunch shearing failure. Haunch shearing failure seems to become more likely as the width of the haunch becomes narrower and the intensity of shear connectors increases, e.g. closely-spaced studs, particularly in pairs. Furthermore, some new decking types such as the KF70® geometry (see Table 2-4) have an additional mini-rib on top of the sheeting rib. This further reduces the distance between the bottom reinforcement mesh and the head of the shear connector, unless higher studs and, in consequence, thicker concrete slabs are provided to fulfill the minimum clear height criteria. A vertical type of haunch reinforcement, similar to the
one provided in solid slab haunch applications (see Figure 2-4), would be far more
effective.

In push-out tests performed by Yuan (1996), no transverse reinforcement below the
head of the stud connectors was provided. Consequently, haunch-shearing failures
were experienced in the specimens with narrow haunches. This mode of failure was
defined as a concrete pull-out failure which is a suitable definition as an inspection of
the failure surface showed that each stud surrounded by a flat “cone” of concrete had
pulled out from the remaining slab. A strength prediction model, assuming that a
concrete pull-out failure in the vertical direction is always accompanied by a
horizontal splitting of the haunch in the transverse direction, was developed in
Johnson and Yuan (1998b). The haunch shearing strength was given as

\[ P_{HS} = \frac{0.6\pi b_{eh} h_{ep} f_t}{(b_{eh} - d_{bs})^2} + 0.56 A_{cone} \sqrt{f_c'} \]

(2-43)

where \( h_{ep} \) is the effective patch height for pulling-out failure determined by equating
the pull-out capacity with the horizontal splitting capacity:

\[ h_{ep} = 2D_c \left( 1 - \frac{0.6\pi d_{bs} D_c}{0.56 A_{cone}} \right) \]

(2-44)

and \( A_{cone} \) is the surface of the projected concrete failure cone whose definition can be
found elsewhere (Johnson and Yuan 1998b). Similar to the stud pull-out failure cone
approaches for secondary composite beams (see Chapter 2.3.2.3), the factor 0.56 in
Equation (2-43) was derived in a purely empirical manner and can be expected to
vary significantly depending on the steel decking geometry investigated and the
approach employed to determined the concrete cone geometry.

2.3.4 Shear connection reinforcing elements

2.3.4.1 Early reinforcing elements

The premature failure modes for secondary and primary composite beams (as
described in Chapters 2.3.2 and 2.3.3) are generally concrete-related and can display
a very brittle behaviour, in particular where pairs of studs are used (Bode and Künzel
1988; Johnson and Yuan 1998a; Mottram and Johnson 1990; Patrick and Bridge 2002a; Patrick and Bridge 2002b). This contradicts the requirement of a ductile shear connection (see Chapter 2.4). None of the design approaches presented in Chapters 2.3.2 and 2.3.3 account for sufficient ductility in a shear connection as generally the maximum connection strength was evaluated with no consideration of the slip capacity.

The first research to place an emphasis on the deformation capacity of the shear connection was performed by Bode and Künzel (1988) where, additionally, the effects of several simple reinforcing measures on the shear connection behaviour were investigated. A U-shaped enhancement component placed around the stud in the pan of a trapezoidal steel decking running transverse to the steel beam was thought to strengthen the concrete bearing zone at the base of the shear connector (Figure 2-16). The steel channel was 400 mm long, 4 mm thick with 40 mm high flanges and was orientated transversely to the steel section. It was tested with various decking geometries and found to increase the maximum shear connection strength between 4% and 38%. As a measure of improved ductility, the lesser of its strength at either 10 mm slip or 90% of the maximum load also increased by 20% to 45%.

Another secondary beam specimen tested included a helix-shaped reinforcing element similar to the one described in Chapter 2.2.4. In this particular test, the shear connection strength could not be improved but its ductility was increased by about 30%. In further tests, the concrete ribs were locally reinforced in accordance with Figure 2-17. For a narrow, dovetailed type of steel decking geometry, both strength and ductility could be increased by around 25%. For a trapezoidal type of steel decking, the strength seemed to be slightly lower, but the specimen behaved in a very ductile manner. Generally, no drops in load were observed before the shear
connections reached slips of 10mm which resulted in an average increase in ductility of about 15% compared to the conventionally reinforced specimens.

Lungershausen (1988) reported two tests where two bars of diagonal rib reinforcement at either side of the shear connection were hooked around the base of a stud (Figure 2-18). In these tests, it was found that the reinforced connection behaved generally stiffer and experienced an increase of about 22% in maximum strength before the load started to drop. It was assumed that the reinforcement reduces the stresses in the concrete bearing zone by anchoring some of the longitudinal stud forces back into the concrete cover slab. However, none of these reinforcing solutions appears to have been put into practice.

2.3.4.2 Waveform reinforcement element for secondary beams

The development of a ide waveform reinforcement element to suppress the brittle effects of rib-shearing failure has been described in detail by Patrick and Bridge (2000). Its individual components are the following (Figure 2-19):

- The waveform longitudinal wires reinforce the horizontal shear surface that passes across the tops of the sheeting ribs. Its ends are anchored in the solid cover part of the concrete slab.
- The transverse wires assist in anchoring the longitudinal wires across the potential failure surface and additionally serve as Type 1 and 2 longitudinal shear reinforcement.
- The lower transverse wires assist in confining the concrete on the bearing side of the stud increasing the connection resistance against rib punch-through failure.

In the commercially available elements, each wire is of 6mm diameter and 500MPa specified yield strength. The spacing of the four longitudinal wires is 150mm, making the nominal overall width 450mm. Each panel is typically 600mm long. The elements are installed once the shear connectors are welded into position and sit directly on the sheeting pan.
Push-out tests on both dovetailed and trapezoidal type of steel decking including this waveform type of reinforcement showed a very ductile behaviour and no signs of horizontal cracking (Patrick and Bridge 2000). However, no stud fractures were experienced in any of the tests and the strength still remained below comparable shear connection strengths in solid slab applications. Composite beam tests on a re-entrant type of steel decking were also conducted in which one side of the beam was conventionally reinforced and the other side included waveform reinforcement elements (Patrick et al. 1995). Once all testing was completed, the slab was cut longitudinally at each end of the beam, very close to the steel beam flange. At the conventional reinforced end, a horizontal crack between adjacent upper corners of the concrete ribs had formed while at the waveform end no such cracks were visible.

The results of the waveform reinforcement tests also formed the basis for the design provisions given in AS 2327.1 (Standards Australia 2003a) for Type 4 longitudinal shear failures. In secondary composite beams, a horizontal Type 4 shear surface has to be reinforced wherever pairs of studs are positioned close to a free concrete edge or where single studs with a transverse slab outstand of less than 600 mm width, measured from the outside edge of the stud, are used. The reinforcement requirements of AS2327.1 (Standards Australia 2003a) to suppress the effects of rib shearing failures are shown in Figure 2-20. The reinforcement must form an angle...
with the horizontal shear surface of between 30° and 90°. Naturally, the waveform reinforcement element presented above can be used as a Type 4 longitudinal shear reinforcement in accordance with AS2327.1 (Standards Australia 2003a). Note that horizontal U-bars passing around the shear connectors of edge beams, as typically required by other standards such as BS5950.3 (BSI 1990) or Eurocode 4 (CEN 2004), are thought to prevent a longitudinal splitting failure of the concrete flange but do not cross this horizontal failure surface and therefore can not reinforce against rib-shearing failure.

**Figure 2-20:** Type 4 longitudinal shear reinforcement requirements given in AS2327.1 (Standards Australia 2003a)

A more generic approach for the design of waveform reinforcement was given in Patrick and Bridge (2002a). If the rib shearing strength in accordance with Equation (2-23) is not satisfied, rib shearing reinforcement is required. With four bars placed on each side of a concrete rib, and ignoring any inclination of the reinforcement to the horizontal plane, the diameter of each wire or bar $d_{wr}$ is determined to be

$$d_{wr} \geq \sqrt{\frac{n_x f_{ys}}{2.8 \phi f_{ysb} (b_{crt} - c_b) k_b}}$$

(2-45)

where $f_{ysb}$ is the design yield stress of the steel reinforcement, $c_b$ the concrete cover measured from nearest edge of the sheeting rib to the centre of bar (see Figure 2-21), $k_b$ the bar anchorage factor, which equals 1.0 if the bars can develop $f_{ysb}$ across the horizontal failure surface, but is otherwise less than 1.0 and $\phi$ is the capacity factor.
for bending of concrete ribs ($\phi=0.8$ in accordance with AS 3600(Standards Australia 2001)).

Figure 2-21: Design of waveform reinforcement to suppress rib shearing failure

(Patrick and Bridge 2002a)

In Patrick and Bridge (2000), it was also suggested that the waveform component could be used in a broader range of situations where horizontal concrete cracking is experienced, such as in negative moment regions of continuous composite beams or composite beams with large web penetrations, or simply as a conventional Type 2 longitudinal shear reinforcement.

2.3.4.3 Ring performance-enhancing device

Preliminary push-out tests have been performed at the University of Western Sydney to investigate the effect of the application of a prototype of the ring performance enhancing device (see also Figure 1-5) in secondary beam applications (Patrick 2002c). Pairs of otherwise identical specimens have been tested, viz. specimens with and without the ring present. The ring devices were normally 40mm high with one specimen having a smaller ring of only 20mm height. They were clipped into position over the 19mm diameter studs once the studs had been welded to a flange plate. The height of the concrete rib was 55mm with the steel decking omitted. Only the free concrete edge on the bearing side of the shear connection was modelled in
these tests (see Figure 2-22). Other variables investigated were the number of studs and the proximity of the studs to the side of the open rib (see Table 2-6).

![Diagram of test set-up](image)

**Figure 2-22:** Test set-up of preliminary ring enhancing device tests

A special test set-up comprising only one solid concrete slab with opposite shear connections was used for these tests as shown in Figure 2-22. The shear forces were applied via two 20mm thick steel plates at either side of the concrete slab. The concrete slabs were 700mm wide. Horizontal movement was prevented by a 20mm thick steel rod at the base of the test specimen.

The results of the tests are summarized in Table 2-6. The load-slip behaviours for some of the specimens are compared in Figure 2-23. It can be seen that the application of the ring device increased the shear connection strength for both single studs and stud pairs. It also improved the ductility of the connection significantly. In the case where the stud was located 95mm from the free concrete edge, which reflects a favourable stud position in a secondary beam concrete rib, a shear connection behaviour similar to a solid slab application was observed, hence the stud was utilized to its full capacity.
Table 2-6: Test specimens of preliminary ring enhancing device tests

<table>
<thead>
<tr>
<th>Spec. No.</th>
<th>No. studs per cross-section $n_x$</th>
<th>Distance to concrete edge $e_b$ [mm]</th>
<th>Ring device</th>
<th>Height of ring device $h_{rd}$ [mm]</th>
<th>Maximum stud strength $P_{max}$ [kN]</th>
<th>Stud strength at 6mm slip $P_{6mm}$ [kN]</th>
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<td>1</td>
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<td>70</td>
<td>No</td>
<td>-</td>
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<td>95</td>
<td>Yes</td>
<td>40</td>
<td>118</td>
<td>114</td>
</tr>
</tbody>
</table>

From these tests it was concluded that the behaviour of the specimens incorporating the ring device was satisfactory and that such devices could be used to increase strength and ductility of shear connections in secondary beam applications, provided that the ring was at least 40mm high. It was also found that the embedment depth of the stud head into the cover part of the slab is very important and studs higher than...
the ones tested (95mm high) could have increased the strength of some of the connections further. During testing, vertical cracks in the concrete ribs just in front of the stud connectors were observed before rib punch-through failures developed. It was believed that the performance of the shear connector could be further improved by including a transverse reinforcement bar in the concrete bearing zone to control this crack. The application of reinforcement spirals as an alternative to the solid ring device was also suggested.

2.4 Slip requirements of shear connections

2.4.1 General

If it is assumed that the shear connection in a composite beam subject to bending prevents the separation between the steel section and the concrete slab at every point along its interface, then both members experience the same rotation \( w' \) at any given cross-section. Full composite interaction between steel beam and concrete member is then achieved if a shear connection of infinite longitudinal stiffness is used. In that case the cross-section has a common neutral axis and no longitudinal slip \( \delta \) develops at the interface between both materials. For the other extreme where the shear connection has zero longitudinal stiffness, no composite interaction develops between steel beam and concrete slab. The capacity of the section is then the sum of the capacities of both members with their individual neutral axes and the longitudinal slip along their interface becomes a maximum. The amount of the slip experienced depends on the positions of the neutral axes in each section and the rotation of the composite beam which itself is a function of the loading configuration and its stiffness. In practice, only very rigid shear connectors such as solid steel blocks effectively create a very stiff shear connection and only require limited slip capacity. But most of the shear connectors available, such as headed stud connectors, are of a flexible type and experience longitudinal slip deformation at the steel-concrete interface when composite interaction is developed. This creates two neutral axes in the beam section with the value of the longitudinal slip experienced being between the two extremes, that of an infinite stiff shear connection and that of a shear
connection with no stiffness (Figure 2-24). Note that the strain distributions, drawn in Figure 2-24 representing an uncracked concrete slab, remain plane.

![Figure 2-24: Strain distribution in a composite beam section for shear connections of different stiffness (after Bärtschi 2005)](image)

### 2.4.2 Elastic design

If linear-elastic behaviours of the shear connection and both members of the composite beam, the concrete slab and the steel section, are assumed, the slip $\delta_c$ in the shear connection can be determined analytically to

$$\delta_c = \frac{\nu_E}{K_c} s_c$$

(2-46)

where $\nu_E$ is the longitudinal shear force per unit length of the composite beam which represents the change of normal forces $dN$ over the beam length $dx$ 

$$\nu_E = \frac{dN}{dx}$$

$s_c$ the longitudinal spacing of the shear connectors and $K_c$ the longitudinal stiffness of the shear connection.

Exact solutions for some particular load cases can be found in the literature, e.g. Newmark et al. (1951). Qualitatively, the distribution of the longitudinal shear force is related to the vertical shear force distribution. In the case of a single concentrated load for example, the distribution would be roughly constant between loading point and support if sufficient stiff shear connections are used. The connectors could then be spaced evenly and each connection would experience approximately the same amount of slip. However, if the shear force distribution is not constant, the shear
connectors would either need to be arranged accordingly to this distribution or, if an even spacing of the shear connectors would be required, the connection which experiences the maximum longitudinal shear would govern the design. The elastic design process can become quite tedious if numerous loading cases need to be considered. For the case of pure linear-elastic design, no plastic deformation capacity of the shear connection is required, hence shear connections which fail in a brittle manner can be used.

### 2.4.3 Plastic design

In building construction, elastic design does not usually lead to economical solutions, consequently composite beams are normally designed assuming plastic stress distributions in the concrete slab and steel section. However, the plastic material behaviours imply stress redistribution along the length of the composite beam and therefore a change in the longitudinal shear force distribution during the course of loading. These changes can not be accommodated by shear connections designed in a linear-elastic manner. The shear connections must rather be able to resist the much increased slip deformations without losing their strength, i.e. a sufficiently ductile behaviour of the shear connections is required.

If a beam is designed in a plastic manner, the load-slip behaviour of the shear connection is usually simplified as ideal-plastic (see also Figure 1-3). The flexible behaviour of the connection is neglected as the slip due to elastic deformation is assumed to be small compared to its plastic deformation. Hence, each shear connection between two critical cross-sections is assumed to transfer the same amount of load regardless of the longitudinal slip distribution. If the strength of the shear connectors governs the composite beam capacity, as is the case in partial shear connection design, the deformation capacity of each shear connection is critical. The inability of one shear connection to accommodate the required slip leads to a consecutive failure of the remaining shear connectors along the beam. Unlike the strength, the shear connection ductility should therefore not be averaged over a number of tests, but rather a minimum slip capacity which was met by all test specimens should be employed.
An analytical determination of the required slip capacity for plastic design becomes very complicated as the interface slip interacts with the longitudinal shear force distribution and the non-linear material behaviours. Many researchers therefore resort to nonlinear numerical analysis (Aribert 1997; Aribert and Al Bitar 1989; Bärtschi 2005; Bode et al. 1994; Bode and Künzel 1988; Johnson and Molenstra 1991). It was found that of the wide array of factors influencing the slip of a composite beam, the most important were the span of the beam and the degree of shear connection. Generally, the slips increase with increasing spans and decreasing degrees of shear connection. The research of Bode and Künzel (1988), for example, analyzed the capacity of a simply supported beam of 5280mm span under single point load assuming a linear-elastic ideal-plastic shear connection behaviour with an elastic stiffness of $K_c=15000\text{N/mm}^2$. A shear connection slip capacity of 10mm was sufficient to ensure that the composite beam reached its predicted bending capacity even for very low degrees of shear connection. However, for slip capacities of only 5mm and partial shear connection, failure occurred before the maximum capacity of each shear connection was reached which effectively reduced the overall capacity of the composite beam. In the case of 2mm shear connection slip capacity, the predicted beam capacity was not even reached for full shear connection. Note that full shear connections also require sufficient ductile behaviour in order to accommodate the stress redistribution due to changes in longitudinal shear. However, a further increase in the degree of shear connection well above the level of full shear connection, $\eta>1$, could have theoretically ensured that the predicted bending capacity of the composite beam would have been reached for the shear connections with limited slip capacity (see also Bärtschi (2005))

On the basis of some of the research work mentioned above, in particular the studies performed by Johnson et al. (1991) and Aribert et al. (e.g. Aribert and Al Bitar 1989, Aribert 1997), limits for the allowable minimum degree of shear connection at a critical cross-section as a function of the beam span and the yield strength of the steel section were derived. In order to apply these limits, a minimum slip requirement for the shear connection needs to be fulfilled which, in accordance with Eurocode 4 (CEN 2004), is set to a characteristic slip capacity of 6mm. In Appendix B of Eurocode 4 (CEN 2004), the slip capacity $\delta_c$ of a shear connection is defined as the
maximum slip measured at the characteristic load level $P_{Rk}$ which can be taken as 90% of the minimum failure load of three or more identical test specimens (see Figure 2-25) or be determined by a statistical evaluation. No specific guidance to designers is given on the stud slip capacity in AS 2327.1 (Standards Australia 2003a) apart from the provision that ‘the shear connection displays ductile behaviour’. Therefore, in the further course of the studies in this thesis, the 6mm slip capacity criteria of Eurocode 4 (CEN 2004) is adopted in order to define ductile shear connection behaviour.

**Figure 2-25:** Determination of slip capacity $\delta_u$ according to Eurocode 4 (CEN 2004)

### 2.5 Methods for determining shear connection behaviour

So-called push or push-out tests have been used extensively in order to determine the strength of shear connectors from as early as the 1930s. They are acknowledged as a substitute for far more expensive and time-consuming full-scale composite beam tests. A standard push test for shear connectors is described in Appendix B of Eurocode 4 (CEN 2004). The dimensions and reinforcement requirements of the standard test specimen are given in Figure 2-26. However, the standard test is not distinctive as it allows an optional recess at the restraint of the slab. It should also be noted that the standard push specimen is also not applicable for specimens incorporating profiled steel decking. No guidance is given in Eurocode 4 (CEN 2004) about the testing arrangements for composite slabs.
Much controversy has arisen over the years about the comparability of the shear connection behaviour in push-tests and composite beam tests. It is generally thought that the shear connection strength derived from push-out tests is a conservative estimate of the strength that the same connection would have in a composite beam application for several reasons:

- The normal compressive stresses induced into the concrete slab due to live and dead loads are absent in push-out specimens.
- The eccentricity which exists in push-out specimens is thought to create uplift effects in the concrete slab and induces additional tensile stresses into the shank of the shear connectors.
- The longitudinal concrete compressive stresses are distributed over a much smaller width in the slabs of push-out tests.
- The stiffness of the concrete slab is much larger in composite beams.
- For the steel decking of composite slabs, transverse anchorages might exist in composite beam applications which are not modelled in push-out tests.

Many researchers therefore introduced modifications to the standard push-out set-up in order to overcome some of these weakening effects. Examples are the tying together of the two concrete slabs (Bode and Künzel 1988; Jenisch 2000; Mainstone and Menzies 1967), the application of a horizontal restraint (Giblings et al. 1992;
Background

Taylor et al. 1970), the use of wider specimens and a concrete recess (Roik and Bürkner 1981; Roik and Hanswille 1983), the inclusion of additional edge reinforcement (see Chapter 4.2) or the application of edge restraints (Hicks and Couchman 2004). While many of these modifications were indeed found to improve the strength of a shear connection, some of them could alter the behaviour of the shear connection altogether by suppressing failure modes that would be experienced otherwise, hence making the push-out test results unconservative and overestimating the slip capacity of the shear connection. On the other hand, the modifications could also introduce new failure modes not possible in composite beam applications. A detailed investigation into the factors influencing the strength and behaviour of various push-out test methods is therefore undertaken in Chapter 3.1.

Generally, it should be kept in mind that, in the case of partial shear connection, changes in shear connection strength only result in comparatively small changes in composite beam capacity. If, on the other hand, a brittle shear connection is used, the connection may fail prematurely before the ultimate beam capacity, calculated on the presumption of ductile behaviour, can be reached. In Figure 2-27, the composite beam capacity $M_{bc}$ is drawn as a function of the degree of shear connection. If, for example, the stud strengths would increase by about 10%, then the degree of shear connection $\eta$ would increase accordingly (e.g. 0.50 to 0.55), but the predicted capacity of the composite beam section would increase at a much lower rate, typically by about 2%-3%. By contrast, if the ductility of the shear connection is not sufficient, the predicted beam capacity can not be reached and, dependent on the stiffness and deformation capacity of the connection, failure of the connection could even occur at bending moments below the moment capacity of the steel section. In that case, the shear connection would be ineffective and, assuming the bending capacity of the concrete slab is negligible, the moment capacity of the steel section alone, $M_s$, would govern the beam capacity. Consequently, when performing push-out tests special care should be taken not to introduce any measures which might increase the slip capacity to a degree which is not feasible in composite beam applications, e.g. the clamping of cracked concrete slabs in tests to ensure continuous load transfer. Hence, the aim of push-out tests should be a conservative modelling of the shear connection deformation, even if it comes at the cost of lower shear connection strength as the impact of the latter on the beam strength is relatively low.
Full-scale beam tests are best suited to investigate the overall behaviour of composite beams and to verify the results obtained from push-out tests. Nevertheless, these tests have to be designed and conducted very carefully in order to deliver realistic results. The modeling of the correct boundary conditions for the beam has a high influence on the reliability of the test results. Determining the strength of the shear connectors from composite beam tests is a complicated and not necessarily a very accurate process. It should be done indirectly by deriving it from the measured strains in the vicinity of the connection rather than by back-calculating it from the applied loads where the normal force distribution along the beam has to be estimated. Figure 2-27 also applies here: small errors in the calculated beam capacity result in much greater errors in the calculated shear connector strength. But even if strain gauges are employed, the steel section will typically start yielding during the test, therefore accurate material properties need to be known. As there are significant amounts of pre-load residual stresses in a hot-rolled steel section (Lay and Ward 1959), the positions of the strain gauges can have a big impact on the recorded yielding point. Moreover, to obtain accurate information about the propagation of the yielding in the steel section, it is necessary to use a sufficient number of strain gauges at each cross-section investigated. It is also a good idea to take the slip readings at several shear connections in order to gather information on the deformation capacity of individual
shear connectors and to be able to compare the findings with accompanying push-out tests. The recorded slips can then also give an indication of the shear connection forces experienced. However, an accurate determination of the behaviour of individual shear connections from composite beam tests is still extremely difficult, hence the recourse to push-out tests. Also, since full-scale beam tests are very expensive, it is important to gather as much data as possible from each test.

2.6 Summary

Shear connections in composite beam applications have a long history of both practical use and extensive research, but many of the strength prediction methods are based on empirical evaluations of tests done decades ago and do not necessarily reflect the real behaviour of those connections. It is considered that the reduction factors taking profiled steel sheeting into account, which are used in Eurocode 4 (CEN 2004) and most national standards around the world, are unsatisfactory and can either over or underestimate the capacity of a connection significantly. A process that considers differentiation between the various failure modes seems to be the more acceptable approach, and several recent proposals to determine the strength for a particular failure mode have been presented. However, an independent evaluation of these methods still needs to be undertaken.

An improved prediction of the shear connection strength is only part of the solution, as none of the approaches account for the lack of ductility that shear connection in composite slabs can experience. But, in particular, the lack of ductility might have the strongest effect on the discrepancy between anticipated and actual behaviour of a composite beam. This becomes most evident in partial shear connection design were a reduced shear connection slip capacity can directly reduce the ultimate bending strength of the entire beam. Most of the problems arising with the use of profiled steel decking are known from previous researchers and measures to improve the behaviour of the shear connection have already been considered. Unfortunately, they have not yet been connected to form a comprehensive model which may also use reinforcing methods to improve the anticipated behaviour.
3 DEVELOPMENT OF AN IMPROVED ONE-SIDED PUSH-OUT TEST RIG

3.1 Influence of test method on shear connection behaviour

Much of the past and present test data used to derive shear connector strength does not comply with the specified standard test method described in Appendix B of Eurocode 4 (CEN 2004). The standard test method is also not applicable to composite slab specimens. The influence of the various test parameters on the shear connection behaviour is examined separately for solid slab and composite slab applications on the basis of push-out test methods found in the literature and subsequently considered in the development of a new push-out test rig.

3.1.1 Solid slab specimens

3.1.1.1 Specimen configuration

The double-sided push-out test set-up standardized in BS5400.5 (BSI 1979) and predominantly used by British researchers in the past consisted of only one row of shear connectors and concrete slabs of 300 mm width. The narrow width might result in concrete cracking of the edges of the specimens (Menzies 1971), a failure not possible in internal composite beams with much wider concrete slabs. Additionally, narrower specimens are more likely to experience a longitudinal splitting failure. A study by Oehlers and Johnson (1987) concluded that two rows of shear connectors are required in order to ensure a redistribution of forces between the shear connectors.
in the two concrete slabs. The use of only one row of shear connectors resulted in a mean connector strength of only 86% of the strength of specimens consisting of two rows. It was argued that two rows would establish the mean connector strength of both slabs while one row would only yield the minimum strength. However, in Ollgaard et al. (1971), specimens with only one row of shear connectors were found to experience similar strengths as specimens having two rows of stud connectors. Dividing the results of push-out tests published by Roik et al. (1988) into different groups also amounted to no obvious influence of specimen width or number of rows of shear connectors on the shear connection strength (Figure 3-1). Hence, the cracking of the edges of the concrete slabs in narrow specimens did not seem to have a big influence on the shear connector capacity. No conclusions can be drawn about the influence of the specimen configuration on the deformation capacity of the shear connection as the slip capacity was usually not published in earlier research. However, in order to obtain more of an average stud strength and ensure force redistribution between individual studs, as required in composite beams with numerous shear connectors, it is considered preferable to test more than one row of stud connectors per concrete slab.

![Figure 3-1: Influence of specimen configuration on stud strength](image.png)
3.1.1.2 Reinforcing of specimens

Push-out tests performed by earlier researchers placed none or very little reinforcement in the slabs which led to premature concrete-related splitting failures and much lower shear connection strengths and deformation capacities (Hawkins 1973; Slutter and Driscoll 1965). The influence of transverse slab reinforcement in particular was investigated in Johnson and Oehlers (1981). The test results showed that specimens which included transverse reinforcement generally delivered higher strengths and larger ductility as the propagation of longitudinal splitting cracks was prevented. However, the reinforcement did not suppress the initial development of these cracks. Hence, further increases in the amount of transverse reinforcement did not increase the shear connection strength. The occurrence of the longitudinal splitting cracks could also be prevented by other means such as an increase in the slab width or an increase in longitudinal stud spacing. The transverse reinforcement required in a push-out specimen can be determined in accordance with the longitudinal reinforcement requirements for shear connections in composite beams as provided by the various standards and codes (e.g. AS 2327.1 (Standards Australia 2003a) or Eurocode 4 (CEN 2004)).

The standard push-out specimen according to Eurocode 4 (CEN 2004) also specifies vertical stirrup reinforcement at the edges of the concrete slab (Figure 2-26) which is aimed at preventing any premature cracking failures of these edges. However, in the solid slab tests described in Chapter 4 and 5 where this stirrup reinforcement was generally omitted and the specimens generally consisted of 600mm wide concrete slabs, no such cracking occurred even for specimens with lower concrete strengths. However, such failures could be experienced when shear connections with more studs or higher connector forces are tested.

3.1.1.3 Recess in concrete slabs

The introduction of a recess at the base of the concrete slab similar to the one suggested for the standard push-out specimen (Figure 2-26) was first proposed by Roik and Hanswille (1983). It is thought that the recess distributes the concrete
compressive forces transversely across the full width of the concrete slab in order to simulate the wider slabs in composite beams. On the contrary, some researchers argue this recess causes additional bending effects in the concrete slab similar to a deep concrete beam application which might initiate vertical bending cracks at the recess (Jenisch 2000). However, these vertical cracks can easily be contained by providing sufficient transverse reinforcement which prevents any further propagation into the concrete slab. To illustrate the influence that a recess has, simple strut and tie models for the standard push-out tests with and without the optional recess are drawn in Figure 3-2. The recess increases the angle of the diagonal compressive strut from around 24° to 35°. The resulting increase in the transverse tensile stresses at the base of the specimen is significant, but the transverse reinforcement normally provided as part of the longitudinal shear design should be sufficient to prevent any vertical cracking failures of the concrete slab.

Tests undertaken by Roik and Hanswille (1983) showed that the inclusion of the concrete recess hardly changed the stud capacity, but it resulted in significantly larger deformation capacities, probably because the free span does reduce the vertical stiffness of the concrete slab. As this effect does not exist in a composite beam and as an accurate prediction of the slip capacity is very important (see Chapter 2.5), it is suggested not to use a recess when testing internal solid slabs. However, in
applications where longitudinal splitting failures might be expected, as for example in haunched cross-sections, a recess should preferably be provided as it is important to accurately model the expected transverse tensile stresses in these applications.

3.1.1.4 Specimen preparation

Until the late 1980s, it was common practice to either cast the specimens in an upright position (some used an inverted position) or to cast each side in the horizontal position but on consecutive days which does lead to different concrete properties within a single specimen. A study on the effect of the concrete-placement direction on the shear connection behaviour (Hiragi et al. 1981) showed that although the ultimate strength of the connector seems to be similar for each pouring direction, specimens which were cast in an upright position experience significant larger deformation thus overestimating the slip capacity of the connection. The larger deformations are attributed to the bleeding of the concrete in the bearing zone of the stud connectors. Casting in the vertical position could even result in voids forming at the stud-concrete interface (Ollgaard et al. 1971). In another study (Kuhn and Buckner 1986), it was found that, apart from reducing the stiffness, the concrete placement also reduced the shear connection strength by around 30%. It is therefore strongly recommended to cast the slab in a horizontal position from the same batch of concrete, as is done for composite beams in practice.

3.1.1.5 Horizontal restraint

The eccentricity which is created due to the push-out test set-up is probably the point of most controversy as neither the amount of horizontal force which balances this eccentricity nor its location are known precisely. The base of the concrete slab is usually embedded in gypsum or mortar and placed on the testing floor which creates a frictional resistance against horizontal movement. In Hicks and McConnel (1996), it was observed that the use of roller bearings instead of the frictional base resulted in significantly lower shear connection strength (Figure 3-3). The reason is obvious: The horizontal uplift forces which are created by the eccentricity of the specimen and
Development of an improved one-sided push-out test rig

which were previously partly balanced by the frictional resistance at its base are now solely induced as tensile forces into the shanks of the shear connectors which reduces their shear capacity. However, the frictional resistance in the standard set-up still seems to allow horizontal movement at the base of the concrete slab, i.e. some uplift to occur between the steel beam and the concrete slab at the base of the specimen. In order to balance these uplift forces, contact forces are thought to develop in the upper regions of the specimen between the concrete slab and the steel section. The height of this contact zone were found to depend on the stiffness of the concrete slab and the amount of separation experienced at the slab base (Mainstone and Menzies 1967). Research has already shown that if separation at the base of the slab is restricted the stud strength can be increased. In Mainstone and Menzies (1967), the two concrete slabs in a push-out specimen were tied together with the help of horizontal steel bars which resulted in an increase in the shear connection strength of around 20% as shown in Figure 3-3. At ultimate load, separation at both the top and the bottom of the concrete slabs was now observed. A stiffer steel rod, as for example suggested in Roik and Hanswille (1987), might completely restrict uplift at the base of the slab. A novel single-sided push-out test set-up was developed by Döinghaus (2002). There, the effect of eccentricity was minimized by tying the slab down and additionally by placing the vertical restraint at a position close to the steel section. Tests showed that the strength of the shear connections also increased about 20% compared to the standard push-out test (Figure 3-3).

![Figure 3-3: Influence of horizontal restraint on shear connection strength](image)

\[ P_e/P_{standard} \text{ for different horizontal restraints} \]
The method of restricting separation has also drawn some criticism. Leonhardt (1988) remarks that if a very stiff horizontal restraint is used, a compressive strut develops directly between restraint and the base of the studs which results in significant compressive stresses in the bearing zone and shank of the studs, in particular in the bottom rows (see Figure 3-4). This effect could increase the shear connection strength above that experienced in composite beams where this effect is not apparent.

In Oehlers and Johnson (1987), push-out tests were undertaken in order to study the influence compression and tension have on the shear connection strength using a special test apparatus as shown in Figure 3-5. The amount of normal forces $T_c$ in the shear connection could be regulated by adjusting the position of the top strut and the knife edge support. If this support was brought below the level of the steel flange compressive forces could be introduced into the stud connection. The test results revealed that compressive forces in the vicinity of the studs increased the shear connection capacity significantly whereas the application of small tensile stresses to
the stud connectors did not have much effect on the capacity (Figure 3-6). Note that the trendline in the graph differs from the one given in Oehlers and Johnson (1987) in that the stud capacity at low tensile forces is assumed to remain constant. The reason for the significant strength increase under compressive loads lies probably in the activation of friction and bond of the concrete surrounding the shear connector. The normal concrete forces may also be able to increase the strength of the triaxial compressive zone which develops at the bearing side of the shear connector and, in consequence, reduce the bending deformation at the base of the stud.

![Graph](image.png)

**Figure 3-6**: Influence of axial stress on shear connection strength (according to test results reported by Oehlers and Johnson (1987))

Normal compressive stresses in the vicinity of the shear connection do not generally appear in composite beams, hence they should not be activated during testing. On the contrary, uplift between concrete slab and steel beam can be expected in simply supported composite beams (Aribert and Abdel Aziz 1985; Chapman and Balakrishnan 1964; Robinson and Naraine 1987) which inevitably results in tensile forces developing in the shank of the stud connectors. Therefore, suppressing all uplift effects in push-out specimens results in unconservative estimates of the shear connection strength experienced in composite beams. However, the extent to which uplift is experienced in beam applications and consequently the amount that should
be permitted in push-out tests is not quite clear and still requires further investigation.

### 3.1.1.6 Normal forces on specimen

In Rambo-Roddenberry et al. (2002), the effect of applying a compressive normal load to the steel-concrete interface in the order of 10% of the longitudinal shear force using the clamping device shown in Figure 3-7 was studied. In the tests, the normal force was first applied in increments followed by an increase in the vertical shear force to regain the 10% ratio. However, it was not reported how the application of the longitudinal shear influenced the horizontal force and if the normal force was readjusted. Specimens, which were otherwise identical but had steel sheeting placed between the steel-concrete interface, were also included in the study. Some of these tests additionally had the sheeting greased to further reduce the influence of friction. The results of this study are summarized in Table 3-1.

![Figure 3-7: Test apparatus to apply normal force (Rambo-Roddenberry et al. 2002)](image)
Table 3-1: Influence of normal force and friction at steel-concrete interface (Rambo-Roddenberry et al. 2002)

<table>
<thead>
<tr>
<th>Push-tests configuration</th>
<th>Normal Force (% of shear force)</th>
<th>Stud capacity $P_{max} / P_{standard}$</th>
<th>Slip capacity $\delta_u$ in (mm)</th>
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</thead>
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<td>Steel-concrete interface (3 tests)</td>
<td>0</td>
<td>1.00</td>
<td>9.5</td>
</tr>
<tr>
<td>Steel-concrete interface (3 tests)</td>
<td>10</td>
<td>1.14</td>
<td>10.3</td>
</tr>
<tr>
<td>Sheet steel at interface (3 tests)</td>
<td>10</td>
<td>0.93</td>
<td>8.1</td>
</tr>
<tr>
<td>Greased sheet steel at interface</td>
<td>10</td>
<td>0.97</td>
<td>7.8</td>
</tr>
</tbody>
</table>

The application of 10% normal forces increased the shear connection strength about 14% and the slip capacity slightly. The results indicate that the normal force activates bond and friction at the steel and concrete interface and that the beneficial effects of this normal force are removed if the coefficient of friction is reduced through the use of sheeting and/or greasing. The test procedure contradicts the requirements of the standard push-out test, as given in Eurocode 4 (CEN 2004), where friction should not be taken into account when determining the connection strength. The assumption made in Rambo-Roddenberry et al. (2002) that a normal load of this magnitude generally exists in shear connections in composite beams can also not be justified as a large proportion of the applied load could consist of single loads which, to make matters worse, could be directly applied to the steel section as is generally the case in primary beam applications. In cases where shear connectors are spaced closely together or in groups, the self-weight of the slab would also account for a normal load significantly less than 10% of the longitudinal shear capacity of the studs. Therefore, it is suggested that normal forces should not be applied in push-out tests unless the permanent existence of such a force can be ensured for the application investigated.

3.1.2 Composite slab specimen

3.1.2.1 Specimen configuration

Numerous researchers (Fisher 1970; Hawkins and Mitchell 1984; Lloyd and Wright 1990; Mottram and Johnson 1990) have shown that the width of the specimen can
have a profound impact on the shear connection strength and behaviour, and that the width of the concrete slab should be wider than 600mm if the steel decking is running transverse to the steel section and an internal beam is to be simulated. The results of push-out tests performed by Lloyd and Wright (1990) on specimens with different slab widths are shown in Figure 3-8. It can be seen that, depending on the width of the specimens, the failure modes changed from rib shearing to stud pull-out. The strength for the narrow specimens was slightly lower, but the differences remained comparatively small. The impact of the specimen width on the shear connection behaviour is studied in more detail in Chapter 4.5.2.6.

Figure 3-8: Influence of concrete slab width on shear connection strength of secondary beam tests performed by Lloyd and Wright (1990)

Specimens with a different number of concrete ribs, viz. two ribs compared to three concrete ribs, did not display significant differences in shear connection strengths (Lloyd and Wright 1990). In Robinson (1988), it was suggested to test only one row of shear connectors per concrete slab in secondary beam applications since in earlier tests, the shear connectors in different rows had experienced different deformations and hence it was concluded that the load was not evenly distributed between the rows. However, it should be considered that the lack of redistribution in a test indicates a brittle failure of one row of shear connectors before the second row of
connectors can reach its full capacity which would make the entire connection unsuitable for plastic design. One of the main purposes of the push-out test should be to identify such brittle behaviours. Therefore, it is recommended to always test more than one row of shear connectors. Furthermore, in modern displacement-controlled push-out tests, the slip can be monitored well past the maximum load clearly identifying successive stud failures. In Jayas and Hosain (1988), the number of rows of shear connectors in otherwise identical specimens was varied between one and three rows and no significant changes in shear connection strength and behaviour could be observed.

### 3.1.2.2 Reinforcing of specimens

A premature failure mode found in secondary beam push-out tests is the transverse cracking of the concrete slab due to bending effects, starting from its top surface above the decking rib where the concrete depth is the smallest. This failure can be referred to as ‘back-breaking’ or ‘rib rolling fracture’ and can be accommodated when adequate longitudinal slab top reinforcement is provided (Kemp and Trinchero 1996; Lawson 1996; Zaki et al. 2003). Tests reported by Zaki et al. (2003) showed that for a specimen where the top reinforcement was accidentally placed too low in the concrete slab, this type of failure was experienced which reduced the shear connection strength around 35% when compared to similar specimens which had the top reinforcement placed in the correct position.

In Figure 3-9 a grossly simplified model for the longitudinal moment distribution in the concrete slab of a push-out specimen is shown (Kemp and Trinchero 1996). The bending stresses which are experienced in the concrete slab depend on the width of the concrete slab, the width and height of the concrete rib, the inner lever arm $a$ and the height of the solid cover part. In the narrow concrete slabs of push-out test specimens, these cracks usually appear earlier than in the wider slabs of composite beam specimens. However, if sufficient longitudinal top reinforcement is provided, the effects of transverse cracking remain localized and do not influence the load-bearing behaviour of the shear connection where the highest stresses are experienced at the stud-concrete interface and in the concrete ribs. When designing the
longitudinal top reinforcement, it should also be kept in mind that for narrow concrete slabs, a higher amount of longitudinal reinforcement is required to accommodate the bending stresses than for wider slabs.

![Diagram of bending effects in cover part of concrete slab](image)

**Figure 3-9:** Bending effects in cover part of concrete slab according to Kemp and Trinchero (1996)

### 3.1.2.3 Recess in concrete slab

In applications where the steel decking runs parallel to the beam, i.e. in primary beam applications, haunch shearing and longitudinal splitting failures can potentially appear. These failures modes might be suppressed if the specimen is restrained along its entire width. The inclination of the concrete compressive struts might also be lower than in a beam application and hence, the transverse tensile stresses which cause longitudinal splitting failure in the concrete haunch are not modelled accurately (see also Figure 3-2). Therefore, the central recess, given as an optional element for the standard specimen as shown in Figure 2-26, should preferably be employed when testing the shear connection in primary composite beam applications. In accordance with solid slab applications (see Chapter 3.1.1.3), no such recess is deemed to be necessary for secondary beam applications.
3.1.2.4 Horizontal restraint and normal forces on specimen

Similar to solid slab applications, a horizontal restraint increases the strength of the shear connection. Early research (Robins 1967) experienced an increase of 5-12% in strength where the failure mode was rib shearing. Jenisch (2000) tested push-out specimens, which were otherwise identical, with and without steel rods tying the two concrete slabs horizontally together. All specimens consisted of three studs per rib and experienced brittle rib shearing failures. Some of the shear connectors were also equipped with strain gauges which measured the axial forces in the stud shanks. The axial force in the steel rods was also measured. The results are summarized in Table 3-2. It can be seen that the stud capacity increased between 13%-33% when the steel rods were used while the slip capacity, which was generally very low due to the brittle failure modes experienced, was not altered much. Furthermore, the tensile forces which developed in the shanks of the studs, expressed as a fraction of the longitudinal shear force, seem to reduce when steel rods were used. The axial tensile forces in the steel rods were about 10%-20% of the maximum vertical load.

Table 3-2: Influence of steel rods on specimen behaviour in push-out tests performed by Jenisch (2000).

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Stud capacity ( P_r / P_{std} )</th>
<th>Slip capacity (at 0.9( P_{max} )) ( \Delta_{slip} ) in (mm)</th>
<th>Axial force in stud at ( P_{max} ) ( T_c / P_{max} )</th>
<th>Force in steel rod ( T_{max} / nP_{max} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cof339A-no steel rod</td>
<td>1.00</td>
<td>&lt;0.5</td>
<td>0.59</td>
<td>-</td>
</tr>
<tr>
<td>Cof339B-steel rod</td>
<td>1.26</td>
<td>1.3</td>
<td>0.48</td>
<td>0.18</td>
</tr>
<tr>
<td>Cof785A-no steel rod</td>
<td>1.00</td>
<td>1.9</td>
<td>0.63</td>
<td>-</td>
</tr>
<tr>
<td>Cof785B-steel rod</td>
<td>1.33</td>
<td>1.9</td>
<td>0.56</td>
<td>0.11</td>
</tr>
<tr>
<td>Cof1589A-no steel rod</td>
<td>1.00</td>
<td>1.4</td>
<td>0.58</td>
<td>-</td>
</tr>
<tr>
<td>Cof1589B-steel rod</td>
<td>1.13</td>
<td>2.0</td>
<td>0.53</td>
<td>0.15</td>
</tr>
</tbody>
</table>

On the contrary, van der Sanden (1996) tested single studs in secondary composite beams using a newly developed horizontal push-out test set-up with lateral restraints (see Figure 3-10) and measured compressive forces at the underside of the steel plate below the central rib. He also tested a stud connector without head. This particular specimen behaved in a similar manner and reached a comparable strength as the specimens including headed stud connectors. In composite beam applications where
tensile stresses can be expected to develop in the shank of the connectors, the concrete slab and steel section would be expected to separate and the studs without heads might be able to eventually pull out of the concrete slab. Hence, the lateral restraints used in this set-up might suppress a potential failure mode.

**Figure 3-10:** Horizontal push-out test set-up according to van der Sanden (1996)

Similar comments can be made about the use of the clamping device shown in Figure 3-7. In tests performed by Rambo-Roddenberry et al. (2002), specimens where the heads of the studs were omitted also experienced exceptional high strengths without any signs of the studs pulling out of the concrete slab. In addition, the longitudinal beam used applied normal compressive forces directly along the line of shear connectors where potential concrete-related failure modes are expected to originate. Hence, any separation between steel section and concrete slab was effectively prevented. In consequence, failure modes which are characterized by near-horizontal concrete cracks such as stud pull-out, rib shearing or haunch shearing were additionally restrained by the presence of the longitudinal beam. It is recommended that such a normal force device should not be used in a standard push-out test.

Some researchers (Mottram and Johnson 1990; Yuan 1996) used a so-called edge trim, a steel formwork that clamps together the steel decking and the concrete slab at the edges of the specimen, during testing of narrow composite slab specimens. It is thought to prevent rib-shearing and simulate the wider concrete slabs in internal beam applications. A study by Hicks and Couchman (2004) showed that the use of edge trim had a profound impact on the shear connection behaviour. For the tests published, it increased the shear connection strength by around 30% and its slip capacity from around 4mm to 8mm (Figure 3-11). Hence it made a brittle shear connection appear ductile. It is therefore recommended not to use this device when performing push-out tests.
Generally, the use of lateral restraints was found to reduce the tensile forces in shear connections or even introduce compressive stresses and therefore increase the shear connection strength. In accordance with the findings for solid slab test methods (see Chapter 3.1.1.5), it is questionable if similar beneficial effects exist in composite beam applications. In addition, many lateral restraint systems seem to alter the shear connection behaviour by suppressing the free development of horizontal concrete cracks or by imposing an additional lateral compression force to the shear connection. As a result, much improved shear connection behaviours, in particular with regards to the slip capacities, were experienced which would not reflect the shear connection behaviour in composite beam applications. Consequently, the use of any lateral reinforcement device should be carefully considered and eventually be verified by comparison tests. On the other hand, tests performed without these devices can be assumed to provide conservative estimates of the shear connection behaviour.

3.1.3 Summary

It was shown that the push-out test method can have a big impact on the shear connection behaviour and, against popular belief, does not necessary provide a
Development of an improved one-sided push-out test rig

conservative estimate. In particular, where concrete-related failure modes might appear, the use of appropriate boundary conditions is crucial as otherwise potential failure surfaces are not free to develop and both strength and slip capacity might be increased beyond what is possible in a composite beam application. In this context, the use of lateral restraints which has become a very popular feature in recent years should be critically reviewed as, in some cases, it seems to completely alter the behaviour of the shear connection (see for example Figure 3-11). Therefore, it is considered that the application of such measures should be verified by composite beam tests. A normal compressive force should only be applied to test specimens where a similar force can be ensured, at any given time, in the composite beam application investigated as additional bond and frictional forces at the steel-concrete interface are activated and compressive forces are induced into both the shear connection and the stud-concrete bearing zone.

3.2 Initial one sided test set-up

3.2.1 General

A one-sided push-out test set-up similar to the one described in Patrick et al. (1995) was initially used for testing. One sided push-out tests have the advantage of reducing the experimental costs significantly by abolishing the inefficient duplication of two identical slabs as only the weaker of the two slabs is tested in double-sided tests. It also overcomes the additional labor required to split the steel section for casting both slabs simultaneously in a horizontal position and to subsequently rejoin these sections before testing. Hence, one-sided push-out tests are an economic alternative to the double sided method and have been used widely by numerous researchers without significant impact on the shear connection behaviour (Hicks and McConnel 1996; Iyengar and Zils 1973; Patrick et al. 1995; Roik 1980; Taylor et al. 1970).
3.2.2 Push-out test rig and instrumentation

The push-out test rig used consisted of: a 1000kN actuator mounted in a loading frame placed above the test specimen; a loading beam placed on the slab; a steel contact plate to locate the steel section and provide reaction to the applied load; and a frame to provide lateral restraint to the top of the specimen during loading. The test-rig is shown in Figure 3-12. Immediately prior to testing, the steel flange of the specimen, which typically was chosen to be a sufficiently thick flat steel plate, is welded to a segment of a rectangular hollow section tube (RHS) to represent the steel beam section. The specimen is then bolted onto the steel base plate. A vertical load is applied to the end of the slab through the horizontal loading beam using the actuator. This system allows for some rotation of the specimen during testing. The end loading pattern of the loading beam, consisting of two steel plates, can be designed to only load specific areas of the concrete slab, if required, as shown in Figure 3-12.

![Figure 3-12: Initial vertical one-sided push-out test set-up](image-url)
As the line of action of the applied vertical load to the slab is eccentric to the reaction at the base of the steel beam in the one-sided set-up, the lateral horizontal movement at the loaded end of the specimen is prevented by a roller which still allows free vertical movement. Consequently horizontal forces are generated at the roller as the vertical load is applied. These are measured during the test by a 200kN load cell attached to the roller.

The positions of the Linear Potentiometers (LPs) are also shown indicatively in Figure 3-12. Four LPs are positioned vertically to measure the longitudinal slip between the slab and the steel beam. They are attached to mounting brackets tack-welded to the steel section. The tip of the transducer is centred on a plate attached to a steel rod embedded in the concrete slab. Where steel decking is used, holes are cut into the decking prior pouring. These are at the position of the steel rods in order to allow relative movement between decking and slab. Four LPs are positioned horizontally to measure the separation between the slab and the steel beam which are also attached to the steel section using magnetic bases. Again, holes were cut into the steel decking prior pouring and a flat sheet plate was placed above the hole in the slab to regain a flat even surface for these measurements. An additional LP is positioned vertically at the loaded end of the specimen to measure the overall longitudinal slip between the slab and the steel beam.

### 3.2.3 Limitations of initial test set-up

During the course of testing, some secondary beam specimens failed prematurely when a vertical crack developed directly under the loading beam propagating from the top of the slab to the tip of the nearest rib (Figure 3-13). For some primary beam specimens, an even more dramatic failure was observed where large parts of the concrete cover slab cracked longitudinally above the top layer of reinforcement (Figure 3-14). Large rotations of the loading beam accompanied these failures. These types of failures have not been observed in earlier double-sided push-out tests where the concrete slab bearing zone was usually placed in gypsum or mortar on the testing floor.
The reason for these failures is believed to lay in the insufficient stiffness of the test rig and becomes more evident if its deformations under loading are considered (Figure 3-15). Due to the eccentricity of the test set-up, a horizontal compressive force $H$ is created in the roller. Horizontal displacement measurements of the loading beam showed that the horizontal roller support used was not perfectly stiff but rather acted as a flexible spring. The majority of the horizontal forces experienced in this lateral support are balanced via bond and friction at the concrete slab – loading beam interface. As the lateral support is flexible, the horizontal forces create lateral movement of the specimen. In Chapter 3.1.1.5, it was shown that similar lateral movements are also experienced in double-sided push-out specimens. However in this set-up, the lateral stiffness is further reduced by the horizontal flexibility of the RHS tube which in return increases the horizontal movement of the specimen. The horizontal specimen movement on the other hand creates a rotation of the loading beam. Measurements during testing showed a horizontal movement at the base of the loading beam of around 3-4mm in secondary beam tests while the top of the loading beam remained more or less in position due to the horizontal restraint provided by the vertical load. The rotation of the loading beam is thought to lead to an uneven transfer of the vertical bearing loads into the concrete slab whereby the larger compressive stresses are concentrated towards the inner regions of the slab. The
outer regions of the slab would consequently experience low levels of compressive stresses which may not be able to accommodate the horizontal forces created at the concrete slab – loading beam interface and eventually may lead to the development of a vertical separation crack in the concrete slab similar to the one shown in Figure 3-13. The failure shown in Figure 3-14 is more of a global nature, but is believed to be of a similar origin. This failure was exclusively experienced in primary beam specimens with a large number of studs where the applied vertical loads were significantly higher as were the lateral movements of the specimen and the rotations of the loading beam.

![Diagram](image.png)

**Figure 3-15:** Deformation of push-out tests under vertical load $P$

Initial solutions for remedying these premature failures were a widening of the test specimens, an increase in the stiffness of the RHS steel section or measures to prevent rotation of the loading beam. However, after careful consideration, it was decided to develop a complete new test set-up which allows more versatility and a wider range of specimens to be tested.
3.3 New horizontal push-out test set-up

3.3.1 Aim

When designing the novel test set-up, the following criteria had to be implemented:

- use of one-sided specimens as this is the most economic way of testing
- high lateral stiffness of steel section
- suitable for a wide range of push-out specimen dimensions
- high load capacity
- easy installation and removal of specimen with minimal material waste
- adjustable concrete slab restraint with optional recess
- options of varying slab restraint conditions and application of normal forces or transverse bending moments
- measurements of all reaction forces

It was decided that these criteria could best be met with the use of a horizontal test rig as it allows greater variety of slab dimensions and provides a very stiff lateral restraint for the steel section. It was furthermore deemed beneficial to load the steel section in tension as this reflects the conditions in a composite beam more realistically and enables the testing of a wider variety of top flanges. In particular, thin top flanges can experience a potential buckling failure in conventional test set-ups. This type of failure would typically not occur in beam applications where the compressive forces in the top flange usually remain insignificant.

3.3.2 Testing rig and features

The new push-out test rig is shown in Figure 3-16. It consists of two horizontal 1000kN actuators mounted in a stiff vertical test frame which apply the loads to a welded loading beam section. The loading beam is connected to two horizontal steel bars of 75mm diameter which transfer the loads via two 100t centrehole compression load cells to a steel section. This welded steel section consists of a base plate, two RHS sections welded together, a web steel plate and a 20mm thick top flange. It is
Development of an improved one-sided push-out test rig

positioned on two Teflon sliding bearings to allow horizontal movement of the section while preventing vertical uplift. A pre-holed connection plate welded to the specimen is mechanically locked into the sliding steel section. Additionally, the connection plate is bolted to the pre-holed top flange of the section in order to ensure sufficient longitudinal shear capacity. After testing, the specimen and the connection plate can be easily removed from the test rig. The connection plate is then ground off the specimen and can be reused several times, minimising material waste during testing. The construction also allows simulation of any type of beam flange by inserting intermediate steel elements between the specimen and the connection plate.

Top View

![Top View Diagram]

Side View

![Side View Diagram]

**Figure 3-16:** New push-out test rig

The horizontal movement of the test specimen is restricted by steel plates attached to a transverse beam section. The end restraint of each specimen, i.e. the two steel plates, can be designed to only load specific areas of the concrete slab simulating the concrete compression zone or to create a recess. The horizontal restraining forces are
then transferred via five rollers to a hardened steel plate attached to the test frame. The rollers ensure uniaxial support and make the line of action adjustable.

As the line of action of the applied horizontal load to the steel section is eccentric to the reaction at the centreline of the restraining patterns, and as lateral vertical movement at the restrained end of the specimen is prevented by friction of the restraining plates, vertical forces are generated. A vertical restraint is attached to both flanges of the transverse beam in order to prevent uplift and rotation of the transverse beam. Four steel plates, two on each flange, were therefore bolted to the transverse beam section. These plates were chosen to be only 5mm thick and 100mm wide in order to minimise the transfer of horizontal loads via shear. The vertical forces are measured during the test by four calibrated Wheatstone-bridges attached to each steel plate of the vertical restraint assemblies as shown in Figure 3-16. The measurements of these uplift forces makes it possible to determine the exact support reaction of the specimen, which is not possible in any conventional push-out test rig. The vertical restraints can be removed in order to investigate the effect of uplift on the shear connection or, alternatively, the pair of vertical restraining plates may be replaced by a single plate in order to allow a free rotation at the end of the specimen similar to the initial test set-up.

Specimens up to 1800mm in width, 2200mm in length and 300mm in height can be attached to this test rig. It is designed for a maximum load of 1500kN which is the equivalent of 12 full strength 19mm diameter stud connectors. The forces can be cycled between zero and maximum load. The shear connection slips and concrete slab uplifts are measured by attaching linear potentiometers to the sliding beam section.

### 3.3.3 Amendments to test rig

During the course of the first test series, several amendments had to be made to the new test set-up.
Amendment of lock mechanism

During testing of the first specimen (HS01-P05), the connection plate started to bend upwards behind the lock mechanism due to the imposed bending moment. An additional vertical support consisting of a horizontal RHS-section and two vertical steel plates was bolted to both sides of the sliding beam in order to prevent any uplift of the connection plate (Figure 3-17). This additional support has to be removed before installing or removing the connection plate.

![Figure 3-17: Amendment of lock mechanism](image)

Strengthening vertical restraint

During testing, it was found that the vertical restraint was not sufficiently stiff but rather allowed some movement of the transverse beam. Figure 3-18 shows the significant rotation the transverse beam experienced during testing. Each vertical restraint consisted of two 5mm thick steel plates and, from the vertical reaction measurements, it became clear that some of the restraints had passed their theoretical buckling capacity. In consequence, the 5mm thick plates were increased to an overall thickness of 10mm, which, as a drawback, increases the horizontal stiffness of these
plates. However, the stiffness still remains negligibly low compared to the stiffness of the transverse beam section.

![Figure 3-18: Rotation of transverse beam during testing](image1)

![Figure 3-19: Longitudinal movement of steel decking](image2)

However, even after these plates were restrained, significant uplift and rotation of the transverse beam was measured (up to 4mm on one side). It was found that the horizontal screws connecting the plates with the Wheatstone-Bridge assemblies experienced significant slips under loading. Consequently, the screw pairs at the base of the vertical restraints were replaced by welds in order to prevent these slips.

**Thickness restraining plates**

The thickness of the restraining plates also needed to be increased as the steel decking in primary beam tests eventually separated from the concrete slab and moved horizontally with the steel section until it touched the transverse beam (Figure 3-19) which led to invalid measurements. Hence, in primary beam tests, it is advised to closely monitor the movement of the steel decking especially when slips of more than 30mm occur.

**3.3.4 Comparison with initial test rig**

As shown in Chapter 3.3.3, the development of forces in the vertical restraints causes vertical movement of the transverse restraining beam and hence, it leads to a vertical
movement of the restraint end of the concrete slabs. The amount of vertical movement experienced depends on the vertical restraining force which itself was found to be closely related to the applied longitudinal shear force. In consequence, the uplift of the specimen increased with the number of stud connectors tested. In the tests conducted so far, it was found that the vertical movement of the transverse beam at maximum load was about 1mm for secondary beam specimens including two single studs, about 1.5mm for secondary specimens including two stud pairs and 2mm for primary beam specimen with six studs. In comparison, the initial test set-up experienced lateral movements at the restraint end of about 3-4mm when two single studs in secondary beam applications were tested. Hence, the aim of a substantial higher lateral stiffness is fulfilled with the new test set-up. In consequence, failures similar to the one shown in Figure 3-13 were not experienced in any of the tests performed to date. No data has been found for the lateral movement at the base of double-sided standard push-out tests. However, in Chapter 3.1.1.5, it was shown that a small amount of lateral movement is also inevitable in double-sided specimens if no specific horizontal restraints are provided, and that such a movement should be tolerated in order to avoid the development of lateral compressive forces in the vicinity of the shear connection.

![Figure 3-20: Comparison of primary beam specimens between the new and initial test set-up](image-url)
A comparison test of a haunched primary beam specimen identical to the one which experienced the dramatic slab failure shown in Figure 3-14 (DEC-HM01) was performed (RIB5-SDM7). The failure was not as dramatic as for the earlier specimen and its shear connection strength increased slightly (Figure 3-20). However, the specimen still experienced extensive cracking and lateral separation of the concrete cover slab in the vicinity of the recess (Figure 3-21). The same types of failure were also experienced for primary beam specimens of a similar lay-out which were tested using the initial test set-up (DEC-HM02 and HM03) and which reached comparable shear connection strengths but displayed a far more ductile behaviour (Figure 3-20). Hence, this type of failure seems to be rather initiated by the recess provided in the concrete slab restraint and the associated deep beam effects induced into the specimen than by a too flexible test set-up as initially thought. More comparison tests would be desirable to investigate this type of failure, its impact on the shear connection ductility and measures to suppress this failure in push-out test specimens as it is not believed to appear in composite beam applications.

![Figure 3-21: Slab failure of specimen RIB5-SDM7](image)

In general, it can be said that the new test set-up has a much larger range of applications and versatility compared with the initial test set-up. It is also able to overcome most of the limitations experienced in the use of the initial test set-up. Apart from the failure displayed in Figure 3-21, only shear connection related failure
modes have been observed in the 43 push-out tests done to date. Its single-sided horizontal arrangement makes testing very economical and user-friendly while still providing a high load capacity.
4. PUSH-OUT TESTS – SECONDARY BEAM APPLICATION

4.1 Introduction

The lack of ductility in shear connections experiencing premature concrete-related failure modes cannot be overcome by further downgrading the stud strength. The most effective way to improve the ductility for a given concrete rib geometry is rather to reinforce the connection against these types of failures. For closed or re-entrant types of steel decking geometry, a waveform type of reinforcement has been found to significantly improve the behaviour of shear connection when located close to a free concrete edge. For these types of applications, the full solid slab stud strength of the shear connectors can generally be applied in accordance with AS 2327.1 (Standards Australia 2003a). As trapezoidal types of steel decking geometries have only recently started to be manufactured in Australia, no thorough investigation into the beneficial effect of this type of reinforcement on the shear connection behaviour has yet been undertaken. Preliminary tests by Patrick (2002c) on a ring type stud performance-enhancing device have also indicated a significant improvement in shear connector strength and ductility when the connectors are placed in steel decking with wide open ribs (see also Chapter 2.3.4.3). However, the application of this device for shear connection incorporating trapezoidal steel decking types still requires further verification.

The various test series presented and discussed in the following have been undertaken over the course of the last five years by the Construction Technology and Research Group at the University of Western Sydney (UWS). Their main purpose
was to obtain a more thorough understanding of the weakening effects that trapezoidal steel decking has on the shear connection behaviour and to systematically investigate the beneficial effects that the application of various types of novel reinforcing elements can have. As many of the tests were related to actual projects, an emphasis was laid on the inclusion of readily available commercial products in the tests such as the trapezoidal steel decking types of KF70® by Fielders Australia or W-Dek® by Lysaght (see Table 2-4) in combination with a slightly modified type of the waveform-shaped shear reinforcement element DECKMESH™ by OneSteel. During the course of the testing, various prototypes of the shear connection performance-enhancing component were applied. For the earlier tests, the ring device StudRing™ was used while for the more recent tests, a newly developed spiral device was applied. A novel horizontal test rig, as described in Chapter 3.3, was also introduced in that period.

The various test series, their results and initial observations are summarized in chronological order in Chapters 4.2 and 4.3 to better reflect the particular development stages of the products. As references, shear connections in solid slab applications were also tested. Apart from providing information on the strength and behaviour that similar connections would experience in solid slabs, they are also used to gather additional information for solid slab applications themselves. More detailed information about each particular test series can be found in the individual test reports (Ernst and Wheeler 2006a; b; c; d; e; g; h; i).

As already described in Chapter 2.3.2, the actual shear connection behaviour is closely related to the failure mode it experiences. The various failure modes which appeared during the course of the test series are described in detail in Chapter 4.4. Both the solid slab and the secondary slab test results are then assessed and analysed in detail in Chapter 4.5, also taking into account further test results published elsewhere.
4.2 Test programs and specimens

4.2.1 Shear connection in KF70® geometry

4.2.1.1 General

A total of 35 push-out tests on a KF70® type of steel decking in 5 different test series were undertaken. All tests were performed using the one-sided vertical test rig as described in Chapter 3.2. The limitations described therein for this test rig apply. Every specimen was 600mm wide, 900 mm long and 130mm high including two full concrete pans. They consisted of two rows of 19mm diameter stud shear connectors, one in each full concrete rib, fast-welded through the steel decking to a 20mm thick and 100mm wide flange plate in all but one test where the sheeting was pre-holed and the studs were welded directly to the flange plate. The stud placements were in the central position in ribs without a lap joint and 30mm from the centreline in the designated longitudinal direction in the ribs containing a lap joint. The specimens further included a front and back half concrete-rib to ensure higher rigidity. A typical specimen is shown in Figure 4-1.

![Figure 4-1: Typical push-out test specimen including waveform reinforcement element and ring enhancing device](image)

In the longitudinal direction, the specimens were reinforced using a top layer of nine 10mm diameter longitudinal reinforcement bars at 60mm transverse spacings. This is a significantly higher amount of longitudinal reinforcement than generally used in
composite beam applications. It was required in order to prevent a ‘back-breaking’ failure of the cover slab as the rotational stiffness of the wider and longer concrete slab in a composite beam application is much higher (see also Chapter 3.1.2.2). The larger amount of reinforcement at the top of the cover slab is not thought to influence the shear connection behaviour as the largest concrete stresses are typically experienced at the base of the shear connector in the concrete ribs. The top transverse reinforcement provided consisted of six 10mm diameter reinforcement bars to fulfil the longitudinal Type 1 and 2 shear reinforcement requirements given in AS2327.1 (Standards Australia 2003a). It was spaced at 150mm centres, welded to the longitudinal bars and placed 20mm clear of the concrete slab top face.

Where the effect of the waveform reinforcement element was investigated, four longitudinal 6mm diameter reinforcing bars with the geometry shown in Figure 4-2 were welded to six transverse 6mm diameter reinforcement bars on its top and, in some specimens, also to four transverse bottom bars of 6mm diameter at the bottom (two in each rib) in the positions shown in Figure 4-2. The component was then placed on top of the lap joint and tied to the underside of the top mesh (Figure 4-3). In order to avoid rib shearing failures and simulate wider internal beam applications, the specimens without this internal waveform component received similar waveform-like longitudinal reinforcing bars at its edges which left the vicinity of the shear connection unreinforced (Figure 4-4). To provide sufficient transverse reinforcement below the head of the shear connector, these specimens additionally received a bottom mesh of welded reinforcement typically consisting of two diameter 10mm transverse reinforcing bars and two diameter 6mm longitudinal round bars placed on the top of the sheeting ribs.

![Figure 4-2: Waveform reinforcement element for KF70® geometry](image-url)
The ring enhancing devices (StudRing™) were fabricated from a 76mm diameter round steel section (nominal grade 400 MPa) of 1.6mm thickness usually cut to 40mm length, except where stated otherwise. Prior to pouring, these rings were clipped over the head of the stud once the shear connector was welded in position (Figure 4-5). The clipping device is designed to hold the ring in place during the pouring process. A special version with a slotted base that can be fitted partly over small lap joints of the decking, thus reducing the offset of the stud, was used in the pans containing this kind of joint.

The specimens of each individual test series were cast in the horizontal position from the same batch of concrete together with numerous 150x100mm concrete cylinders which were stored alongside the specimen and tested at regular intervals.
4.2.1.2 SRG-P01 series

The first small test series was designed to investigate the shear connection behaviour using the ring enhancing device in combination with the waveform reinforcement element for single stud arrangements. The test variables were the height of the ring enhancing device and the placement of the stud connector in the concrete rib.

Table 4-1: Test program SRG-P01 series

<table>
<thead>
<tr>
<th>Code</th>
<th>Studs (height in mm)</th>
<th>Stud placement</th>
<th>Stud placement¹</th>
<th>Transverse stud spacing (mm)</th>
<th>Cross-wire in concrete ribs²</th>
<th>Waveform element</th>
<th>Ring device</th>
<th>Target concrete strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S3</td>
<td>Singles (100)</td>
<td>Central/favourable</td>
<td>N/A</td>
<td>No</td>
<td>Yes</td>
<td>Yes, 40mm high</td>
<td>30</td>
<td></td>
</tr>
<tr>
<td>S4</td>
<td>Singles (100)</td>
<td>Central/unfavourable</td>
<td>N/A</td>
<td>No</td>
<td>Yes</td>
<td>Yes, 70mm high</td>
<td>30</td>
<td></td>
</tr>
</tbody>
</table>

¹: stud placements in central position in ribs without lap joint and placed 30mm in designated longitudinal direction away from lap joint
²: round 6mm diameter bars welded to base of either edge- or waveform reinforcement element

4.2.1.3 SRG-P02 series

Table 4-2: Test program SRG-P02 series

<table>
<thead>
<tr>
<th>Code</th>
<th>Studs (height in mm)</th>
<th>Stud placement</th>
<th>Stud placement¹</th>
<th>Transverse stud spacing (mm)</th>
<th>Cross-wire in concrete ribs²</th>
<th>Waveform element</th>
<th>Ring device</th>
<th>Target concrete strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>Pairs (100)</td>
<td>Central/diagonal</td>
<td>80</td>
<td>No</td>
<td>No</td>
<td>Yes</td>
<td>30</td>
<td></td>
</tr>
<tr>
<td>S2</td>
<td>Pairs (100)</td>
<td>Central/diagonal</td>
<td>80</td>
<td>No</td>
<td>No</td>
<td>Yes</td>
<td>30</td>
<td></td>
</tr>
<tr>
<td>S3</td>
<td>Pairs (100)</td>
<td>Central/diagonal</td>
<td>120</td>
<td>No</td>
<td>No</td>
<td>No</td>
<td>30</td>
<td></td>
</tr>
<tr>
<td>S4</td>
<td>Pairs (100)</td>
<td>Central/diagonal</td>
<td>120</td>
<td>No</td>
<td>No</td>
<td>Yes</td>
<td>30</td>
<td></td>
</tr>
<tr>
<td>S5</td>
<td>Pairs (100)</td>
<td>Central/favourable</td>
<td>80</td>
<td>No</td>
<td>No</td>
<td>No</td>
<td>30</td>
<td></td>
</tr>
<tr>
<td>S6</td>
<td>Pairs (100)</td>
<td>Central/favourable</td>
<td>80</td>
<td>No</td>
<td>No</td>
<td>Yes</td>
<td>30</td>
<td></td>
</tr>
</tbody>
</table>

¹: stud placements in central position in ribs without lap joint and placed 30mm in designated longitudinal direction away from lap joint
²: round 6mm diameter bars welded to base of either edge- or waveform reinforcement element
³: horizontal restraint at base of concrete slab during testing
The second series focused on the behaviour of shear connections incorporating stud pairs. Factors investigated were the use of the ring enhancing device, the transverse stud spacing and the position of shear connectors in the concrete ribs. Furthermore, the outer face of the bottom part of the concrete slab in specimen S6 was restrained horizontally during testing in order to gather information on the influence that the test method has on the specimen behaviour (Figure 4-6).

![Figure 4-6: Horizontal slab restraint at specimen S6 during testing](image)

4.2.1.4 SRG-P03 series

This subsequent test series completed the previous two series into the ring enhancing device behaviour. For single stud applications, the influence of the ring enhancing device, the sheeting thickness and different concrete strengths were investigated if no waveform reinforcement elements were provided. For stud pair applications, the use of waveform reinforcement elements with and without the ring enhancing device and the influence of transverse cross-wires in the concrete ribs were the main variables. The other variables investigated were the concrete strength, stud height and pre-holing of the steel decking. An accompanying solid slab specimen was also included in this series.
Table 4-3: Test program SRG-P03 series

<table>
<thead>
<tr>
<th>Code</th>
<th>Studs (height in mm)</th>
<th>Sheet thickness (mm)</th>
<th>Stud placement¹</th>
<th>Transverse stud spacing (mm)</th>
<th>Cross-wire in concrete ribs²</th>
<th>Waveform element</th>
<th>Ring device</th>
<th>Target concrete strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SR1</td>
<td>Singles (100)</td>
<td>1.0</td>
<td>Central/favourable</td>
<td>N/A</td>
<td>No</td>
<td>No</td>
<td>No</td>
<td>30</td>
</tr>
<tr>
<td>SR2</td>
<td>Singles (100)</td>
<td>0.6</td>
<td>Central/favourable</td>
<td>N/A</td>
<td>No</td>
<td>No</td>
<td>No</td>
<td>40</td>
</tr>
<tr>
<td>SR3</td>
<td>Singles (100)</td>
<td>1.0</td>
<td>Central/favourable</td>
<td>N/A</td>
<td>No</td>
<td>No</td>
<td>No</td>
<td>40</td>
</tr>
<tr>
<td>SR4</td>
<td>Singles (100)</td>
<td>1.0</td>
<td>Central/favourable</td>
<td>N/A</td>
<td>No</td>
<td>No</td>
<td>Yes</td>
<td>30</td>
</tr>
<tr>
<td>SR5</td>
<td>Pairs (100)</td>
<td>1.0</td>
<td>Central/favourable</td>
<td>80</td>
<td>Yes</td>
<td>Yes</td>
<td>No</td>
<td>30-40</td>
</tr>
<tr>
<td>SR6</td>
<td>Pairs (100)</td>
<td>1.0</td>
<td>Central/favourable</td>
<td>80</td>
<td>Yes</td>
<td>Yes</td>
<td>No</td>
<td>30-40</td>
</tr>
<tr>
<td>SR7</td>
<td>Pairs (100)</td>
<td>1.0</td>
<td>Central/favourable</td>
<td>80</td>
<td>No</td>
<td>Yes</td>
<td>No</td>
<td>30-40</td>
</tr>
<tr>
<td>SR8</td>
<td>Pairs (100)</td>
<td>1.0</td>
<td>Central/favourable</td>
<td>80</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>30-40</td>
</tr>
<tr>
<td>SR9</td>
<td>Pairs (100)</td>
<td>1.0</td>
<td>Central/favourable</td>
<td>80</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>30-40</td>
</tr>
<tr>
<td>SR10</td>
<td>Pairs (100)</td>
<td>1.0</td>
<td>Central/favourable</td>
<td>80</td>
<td>No</td>
<td>Yes</td>
<td>Yes</td>
<td>30-40</td>
</tr>
<tr>
<td>SR12</td>
<td>Pairs (100)</td>
<td>Solid Slab³</td>
<td>300mm crs long.</td>
<td>80</td>
<td>No</td>
<td>No</td>
<td>No</td>
<td>30-40</td>
</tr>
<tr>
<td>SR13</td>
<td>Pairs (130)</td>
<td>1.0</td>
<td>Central/favourable</td>
<td>80</td>
<td>No</td>
<td>No</td>
<td>No</td>
<td>30-40</td>
</tr>
</tbody>
</table>

¹: stud placements in central position in ribs without lap joint and placed 30mm in designated longitudinal direction away from lap joint
²: round 6mm diameter bars welded to base of either edge- or waveform reinforcement element
³: top and bottom layer of reinforcement
⁴: steel decking pre-holed

4.2.1.5 OSR-DM01 series

The purpose of this series was to gather information on the behaviour of the shear connection in conventionally reinforced specimens, the influence of a transverse cross-wire at the base of a concrete rib and the influence of the waveform reinforcement element including transverse cross-wires if stud pairs are used; all at different concrete strengths.
Table 4-4: Test program OSR-DM01 series

<table>
<thead>
<tr>
<th>Code</th>
<th>Studs (height in mm)</th>
<th>Sheet thickness (mm)</th>
<th>Stud placement</th>
<th>Transverse stud spacing (mm)</th>
<th>Cross-wire in concrete ribs</th>
<th>Waveform element</th>
<th>Ring device</th>
<th>Target concrete strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S01</td>
<td>Pairs (100)</td>
<td>1.0</td>
<td>Central/ diagonal</td>
<td>80</td>
<td>Yes</td>
<td>Yes</td>
<td>No</td>
<td>20</td>
</tr>
<tr>
<td>S02</td>
<td>Pairs (100)</td>
<td>1.0</td>
<td>Central/ diagonal</td>
<td>80</td>
<td>Yes</td>
<td>Yes</td>
<td>No</td>
<td>30</td>
</tr>
<tr>
<td>S03</td>
<td>Pairs (100)</td>
<td>1.0</td>
<td>Central/ diagonal</td>
<td>80</td>
<td>Yes</td>
<td>Yes</td>
<td>No</td>
<td>40</td>
</tr>
<tr>
<td>S04</td>
<td>Pairs (100)</td>
<td>1.0</td>
<td>Central/ diagonal</td>
<td>80</td>
<td>Yes</td>
<td>No</td>
<td>No</td>
<td>20</td>
</tr>
<tr>
<td>S05</td>
<td>Pairs (100)</td>
<td>1.0</td>
<td>Central/ diagonal</td>
<td>80</td>
<td>Yes</td>
<td>No</td>
<td>No</td>
<td>30</td>
</tr>
<tr>
<td>S06</td>
<td>Pairs (100)</td>
<td>1.0</td>
<td>Central/ diagonal</td>
<td>80/120</td>
<td>Yes</td>
<td>No</td>
<td>No</td>
<td>40</td>
</tr>
<tr>
<td>S07</td>
<td>Singles (100)</td>
<td>1.0</td>
<td>Central/ favourable</td>
<td>N/A</td>
<td>Yes</td>
<td>No</td>
<td>No</td>
<td>20</td>
</tr>
<tr>
<td>S08</td>
<td>Singles (100)</td>
<td>1.0</td>
<td>Central/ favourable</td>
<td>N/A</td>
<td>Yes</td>
<td>No</td>
<td>No</td>
<td>30</td>
</tr>
<tr>
<td>S09</td>
<td>Singles (100)</td>
<td>1.0</td>
<td>Central/ favourable</td>
<td>N/A</td>
<td>Yes</td>
<td>No</td>
<td>No</td>
<td>40</td>
</tr>
<tr>
<td>S10</td>
<td>Singles (100)</td>
<td>1.0</td>
<td>Central/ favourable</td>
<td>N/A</td>
<td>No</td>
<td>No</td>
<td>No</td>
<td>20</td>
</tr>
<tr>
<td>S11</td>
<td>Singles (100)</td>
<td>1.0</td>
<td>Central/ favourable</td>
<td>N/A</td>
<td>No</td>
<td>No</td>
<td>No</td>
<td>30</td>
</tr>
<tr>
<td>S12</td>
<td>Singles (100)</td>
<td>1.0</td>
<td>Central/ favourable</td>
<td>N/A</td>
<td>No</td>
<td>No</td>
<td>No</td>
<td>40</td>
</tr>
</tbody>
</table>

1: stud placements in central position in ribs without lap joint and placed 30mm in designated longitudinal direction away from lap joint.
2: round 6mm diameter bars welded to base of either edge- or waveform reinforcement element.

4.2.1.6 OBD-HS01 series

The last series on the KF70® geometry investigated the effects that the position of a single stud in the concrete pan and that the use of stud enhancing devices have on the shear connection behaviour if higher shear connectors are used.
### Table 4-5: Test program OBD-HS01 series

<table>
<thead>
<tr>
<th>Code</th>
<th>Studs (height in mm)</th>
<th>Sheet thickness (mm)</th>
<th>Stud placement(^1)</th>
<th>Transverse stud spacing (mm)</th>
<th>Cross-wire in concrete ribs(^2)</th>
<th>Waveform element</th>
<th>Ring device</th>
<th>Target concrete strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S01</td>
<td>Singles (125)</td>
<td>0.75</td>
<td>Central/ favourable</td>
<td>N/A</td>
<td>No</td>
<td>No</td>
<td>No</td>
<td>Yes</td>
</tr>
<tr>
<td>S02</td>
<td>Singles (125)</td>
<td>0.75</td>
<td>Central/ unfavourable</td>
<td>N/A</td>
<td>No</td>
<td>No</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>S03</td>
<td>Singles (125)</td>
<td>0.75</td>
<td>Central/ favourable</td>
<td>N/A</td>
<td>No</td>
<td>No</td>
<td>Yes</td>
<td>30-40</td>
</tr>
<tr>
<td>S04</td>
<td>Singles (125)</td>
<td>0.75</td>
<td>Central/ unfavourable</td>
<td>N/A</td>
<td>No</td>
<td>No</td>
<td>Yes</td>
<td>30-40</td>
</tr>
</tbody>
</table>

\(^1\): stud placements in central position in ribs without lap joint and placed 30mm in designated longitudinal direction away from lap joint

\(^2\): round 6mm diameter bars welded to base of either edge- or waveform reinforcement element

---

### 4.2.2 Shear connection in W-Dek® geometry

#### 4.2.2.1 General

A total of 36 push-out tests in five separate test series were conducted to determine the behaviour of the shear connection in secondary beam applications if the Lysaght W-Dek® geometry is used. All tests were performed using the new horizontal test rig as described in Chapter 3.3. All concrete slabs had an overall depth of 150 mm, were 1050 mm long with two rows of stud shear connectors while the width of the slab was a variable in the tests. The specimens were generally designed to satisfy the requirements of Eurocode 4 (CEN 2004).

The shear connection consisted of 19mm diameter headed studs with a specified overall height of 127mm after welding. They were automatically-welded to the centre of each concrete rib with the exception of four specimens tested in the series CTR-RIB5 where the studs were placed in a diagonal position. Where steel decking was used, it needed to be pre-holed in order to position the studs in the centre of the rib because the W-DEK® geometry includes either a central longitudinal stiffener or lap joint in the pans. Pre-holing of the steel decking is not common practice in
Australia, but it has the major advantage of producing substantially better and far more reliable stud welds with fewer failures. In the composite beam tests recorded in Chapter 7, more than 50% of the studs automatically-welded through the Z350 0.75mm thick steel sheeting did not have a 360 degree weld bead. Around 25% of the studs failed when tested in accordance with AS/NZS1554.2 (Standards Australia 2003b), despite ideal laboratory conditions. It was then decided to remove the studs from the top flange, pre-hole the steel decking and weld new studs directly to the top flange. The same procedure was consequently applied for the accompanying specimens in series CTR-RIB5. In some of the test specimens, several studs had strain gauges fitted internally in their shank to determine the tensile force that developed when the studs are loaded in shear (see also Figure 4-11). The strain gauges were placed in a 2 mm diameter central hole that extended from the top of the stud to 20mm below the underside the head. The holes were sealed with a special adhesive to protect the gauges which were accurately calibrated using the results of stud tensile tests.

Some specimens consisted of plain concrete ribs with the same concrete slab geometry but with the steel decking omitted. This allowed the influence that the steel
sheeting has on the load transfer mechanism to be investigated. When used, the decking (made from high-tensile galvanized G550 steel) was anchored into the concrete slab at both of its ends by cutting the decking into small tabs and bending them over about 135° (Figure 4-7). This was done in order to prevent the decking separating from the concrete ribs, as observed in previous testing, and to simulate the anchorage developed by the decking in a wider internal secondary beam where the decking extends over the concrete slab for a much greater width. However, this high level of anchorage might overestimate the strengthening effect of the decking in shear connections close to a free concrete edge.

Instead of using the earlier ring prototype that was cut from a round steel tube, the stud performance-enhancing device consisted of a 4mm diameter round steel wire that spiralled around the stud. At the bottom of the spiral was a circular retaining loop with an inner diameter of 35mm which prevented excessive lateral movement of the device during the concrete pouring operation but still permitted the base of the device to be fitted over the head of a 19mm diameter stud. The outer diameter of the main spiral was 76mm diameter and was constant for a height of about 60mm above the soffit of the decking, while the pitch of the spiral was less for the bottom 40mm. The spiral diameter then reduced at a constant rate over the remaining height, reaching a minimum diameter of 33mm where it provided a tight fit over the top of the head of the stud (Figure 4-8).

Where the waveform reinforcement element was used, it was designed to provide 20mm concrete cover at the underside surfaces of the concrete ribs while it was
placed directly on top of the lap joints and tied to the top reinforcement to hold it in position. The two geometries used are shown in Figure 4-10. A section through a typical test specimen, including the waveform reinforcement element and the spiral enhancing device, is shown in Figure 4-9.

![Top View - Waveform Reo for Specimen wider than 500mm](image1)

![Top View - Waveform Reo for Specimen 400mm wide](image2)

**Figure 4-10**: Waveform reinforcement elements for W-Dek® geometry

The top reinforcement mesh was placed 20mm below the top surface and was designed to contain sufficient longitudinal reinforcement in order to prevent local bending failure of the concrete slab. Where no waveform reinforcement elements were used, the specimens typically included a bottom layer of reinforcement placed directly on top of the concrete ribs with 10mm diameter transverse bars welded to the underside of the longitudinal 6mm diameter reinforcing bars and extending into the
concrete ribs, in order to provide the maximum possible clear vertical distance between the transverse wires and the underside of the head of the studs. Only in two tests was the bottom layer omitted. This was to investigate the effect this change in detail had on the shear connection behaviour.

In some of the specimens, strains in the concrete, at critical locations across the horizontal plane between the concrete ribs and the solid cover slab, were also measured. For this purpose, each strain gauge was glued onto the smooth side of a piece of brittle masonite previously sealed with polyurethane. The pieces of masonite did not require to be end anchored due to their low tensile strength and the rough surface. The strain gauges were then protected with mastic and, finally, each completed assemblage was tied to the rear transverse reinforcing bar and located in the specified concrete rib of the specimen. Gauges were located at the designated positions transversely from the longitudinal centreline of the specimen, either to its right side (R) and/or left side (L), as indicated in Figure 4-11.

![Figure 4-11: Concrete and stud strain gauges in a test specimen (CTR-RIB1-SW5)](image)

The concrete for the test specimens in each series was poured in the horizontal position from the same batch. Alongside the test specimens, concrete cylinders were cured to determine the concrete compressive strength at the necessary ages. All of
the test specimens and their companion concrete cylinders were kept moist until testing resumed.

### 4.2.2.2 CTR-RIB1 series

The first test series was designed to investigate a conventional reinforced single stud arrangement. Most of the specimens consisted of plain concrete ribs without steel decking which are believed to contribute to a better mechanical understanding of the shear connection behaviour. Some of the specimens were equipped with concrete and stud strain gauges. The variables investigated were the width of the test specimen, the existence of a bottom reinforcement layer and the concrete strength. A companion solid slab specimen consisting of the same stud arrangement was also included to provide the reference shear connection strength and behaviour.

#### Table 4-6: Test program CTR-RIB1 series

<table>
<thead>
<tr>
<th>Code</th>
<th>Slab width (mm)</th>
<th>Studs</th>
<th>Sheet thickness (mm)</th>
<th>Stud placement</th>
<th>Bottom reinforcement</th>
<th>Waveform element</th>
<th>Spiral device</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>SW1</td>
<td>1200</td>
<td>Singles</td>
<td>0.75&lt;sup&gt;1&lt;/sup&gt;</td>
<td>Central</td>
<td>Yes</td>
<td>No</td>
<td>No</td>
<td></td>
</tr>
<tr>
<td>SW2</td>
<td>1200</td>
<td>Singles</td>
<td>0.75&lt;sup&gt;1&lt;/sup&gt;</td>
<td>Central</td>
<td>No</td>
<td>No</td>
<td>No</td>
<td></td>
</tr>
<tr>
<td>SW3</td>
<td>1200</td>
<td>Singles</td>
<td>No sheet</td>
<td>Central</td>
<td>Yes</td>
<td>No</td>
<td>No</td>
<td></td>
</tr>
<tr>
<td>SW4</td>
<td>1200</td>
<td>Singles</td>
<td>No sheet</td>
<td>Central</td>
<td>No</td>
<td>No</td>
<td>No</td>
<td></td>
</tr>
<tr>
<td>SW5</td>
<td>900</td>
<td>Singles</td>
<td>No sheet</td>
<td>Central</td>
<td>Yes</td>
<td>No</td>
<td>No</td>
<td>Concrete and stud gauges&lt;sup&gt;2,3&lt;/sup&gt;</td>
</tr>
<tr>
<td>SW6</td>
<td>600</td>
<td>Singles</td>
<td>No sheet</td>
<td>Central</td>
<td>Yes</td>
<td>No</td>
<td>No</td>
<td>Concrete and stud gauges&lt;sup&gt;2,3&lt;/sup&gt;</td>
</tr>
<tr>
<td>SW7</td>
<td>400</td>
<td>Singles</td>
<td>No sheet</td>
<td>Central</td>
<td>Yes</td>
<td>No</td>
<td>No</td>
<td>Concrete and stud gauges&lt;sup&gt;2,4&lt;/sup&gt;</td>
</tr>
<tr>
<td>SW8</td>
<td>600</td>
<td>Singles</td>
<td>Solid slab&lt;sup&gt;5&lt;/sup&gt;</td>
<td>350crs</td>
<td>N/A</td>
<td>No</td>
<td>No</td>
<td></td>
</tr>
</tbody>
</table>

<sup>1</sup>: steel decking anchored at its transverse edges
<sup>2</sup>: concrete gauges located in the rear side of the rear rib 0, 100mm and, if applicable, 200mm and 300mm transversely from centreline to both sides
<sup>3</sup>: stud gauge placed in shank of rear stud
<sup>4</sup>: stud gauges placed in shank of front and rear stud
<sup>5</sup>: reinforced with welded top and bottom mesh of 6N12 long. and 6N10 transv. reinforcement bars
**4.2.2.3 CTR-RIB2 series**

The purpose of the second test series was the investigation of a conventional reinforced double stud arrangement. A particular emphasis was placed on the influence that the width of the test specimen had on its strength and behaviour. All of the specimens consisted of plain concrete ribs without steel decking. Strain measurements were taken for all specimens; in the shank of the studs as well as in the rear concrete ribs of the specimens.

<table>
<thead>
<tr>
<th>Code</th>
<th>Slab width (mm)</th>
<th>Studs</th>
<th>Sheet thickness (mm)</th>
<th>Stud placement</th>
<th>Bottom reinforcement</th>
<th>Waveform element</th>
<th>Spiral device</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>SWP1</td>
<td>900</td>
<td>Pairs</td>
<td>No sheet</td>
<td>Central @ 80crs</td>
<td>Yes</td>
<td>No</td>
<td>No</td>
<td>Concrete and stud gauges$^{1,2}$</td>
</tr>
<tr>
<td>SWP2</td>
<td>600</td>
<td>Pairs</td>
<td>No sheet</td>
<td>Central @ 80crs</td>
<td>Yes</td>
<td>No</td>
<td>No</td>
<td>Concrete and stud gauges$^{1,2}$</td>
</tr>
<tr>
<td>SWP3</td>
<td>400</td>
<td>Pairs</td>
<td>No sheet</td>
<td>Central @ 80crs</td>
<td>Yes</td>
<td>No</td>
<td>No</td>
<td>Concrete and stud gauges$^{1,2}$</td>
</tr>
</tbody>
</table>

1: concrete gauges located in the rear side of the rear rib 0, 100mm and, if applicable, 200mm transversely from centreline to both sides

2: stud gauges placed in shank of front and rear studs

**4.2.2.4 CTR-RIB3 series**

The third test series was designed to study the influence of the spiral stud enhancement device for both single stud and stud pair applications in narrow and wide specimens. The inclusion of conventional reinforced specimens with stud pairs in both wider concrete slabs and with the use of steel decking was another feature of this series. On some of the specimens, strain measurements were taken in the shank of the stud as well as in the concrete ribs to gain a better understanding of the shear connection behaviour.
Table 4-8: Test program CTR-RIB3 series

<table>
<thead>
<tr>
<th>Code</th>
<th>Slab width (mm)</th>
<th>Studs</th>
<th>Sheet thickness (mm)</th>
<th>Stud placement</th>
<th>Bottom reinforcement</th>
<th>Waveform element</th>
<th>Spiral device</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>SWR1</td>
<td>1200</td>
<td>Pairs</td>
<td>No sheet</td>
<td>Central @ 80crs</td>
<td>Yes</td>
<td>No</td>
<td>No</td>
<td>Concrete gauges²</td>
</tr>
<tr>
<td>SWR2</td>
<td>1800</td>
<td>Pairs</td>
<td>No sheet</td>
<td>Central @ 80crs</td>
<td>Yes</td>
<td>No</td>
<td>No</td>
<td>Concrete gauges³</td>
</tr>
<tr>
<td>SWR3</td>
<td>600</td>
<td>Pairs</td>
<td>No sheet</td>
<td>Central @ 80crs</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
<td>Concrete and stud gauges³,⁴</td>
</tr>
<tr>
<td>SWR4</td>
<td>1200</td>
<td>Pairs</td>
<td>No sheet</td>
<td>Central @ 80crs</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
<td>Concrete gauges³</td>
</tr>
<tr>
<td>SWR5</td>
<td>1800</td>
<td>Pairs</td>
<td>No sheet</td>
<td>Central @ 80crs</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
<td>Concrete gauges³</td>
</tr>
<tr>
<td>SWR6</td>
<td>600</td>
<td>Pairs</td>
<td>1.00¹</td>
<td>Central @ 80crs</td>
<td>Yes</td>
<td>No</td>
<td>No</td>
<td>Stud gauges⁴</td>
</tr>
<tr>
<td>SWR7</td>
<td>1200</td>
<td>Pairs</td>
<td>1.00¹</td>
<td>Central @ 80crs</td>
<td>Yes</td>
<td>No</td>
<td>No</td>
<td></td>
</tr>
<tr>
<td>SWR8</td>
<td>400</td>
<td>Singles</td>
<td>No sheet</td>
<td>Central</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
<td>Stud gauges⁴</td>
</tr>
<tr>
<td>SWR9</td>
<td>1200</td>
<td>Singles</td>
<td>No sheet</td>
<td>Central</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
<td>Concrete and stud gauges³,⁴</td>
</tr>
</tbody>
</table>

¹: steel decking anchored at its transverse edges  
²: concrete gauges located in the rear sides of both ribs 0, 100mm, 200mm, 300mm and 400mm transversely from centreline to left side  
³: concrete gauges located in the rear side of the rear rib 0, 100mm, 200mm, 400mm and, if applicable, 600mm transversely from centreline to both sides  
⁴: stud gauges placed in shanks of front and rear studs

4.2.2.5 CTR-RIB4 series

In this series, the application of the waveform reinforcement element in combination with the spiral stud enhancing device was investigated for narrow and wide specimens, both for single stud and stud pair applications. The influence that the steel decking has on the application of the spiral enhancing device was another parameter investigated. Where the waveform reinforcement element was used, its longitudinal bars were equipped with steel strain gauges glued to the position indicated in Figure 4-12 in order to determine the amounts of vertical axial forces transferred. Companion solid slab specimens with a similar stud pair arrangement were also included. One of these otherwise identical specimens consisted of a Teflon layer placed between the steel-concrete interface in order to investigate the influence of the frictional component on the shear connection strength. Additionally, strain measurements in the shank of the studs were taken on many of the specimens.
### Table 4-9: Test program CTR-RIB4 series

<table>
<thead>
<tr>
<th>Code</th>
<th>Slab width (mm)</th>
<th>Slab</th>
<th>Studs</th>
<th>Sheet thickness (mm)</th>
<th>Stud placement</th>
<th>Bottom reinforcement</th>
<th>Waveform element</th>
<th>Spiral device</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>SWD1</td>
<td>400</td>
<td>Singles</td>
<td>0.75&lt;sup&gt;1&lt;/sup&gt;</td>
<td>Central</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
<td>Stud gauges&lt;sup&gt;3&lt;/sup&gt;</td>
<td></td>
</tr>
<tr>
<td>SWD2</td>
<td>1200</td>
<td>Singles</td>
<td>0.75&lt;sup&gt;1&lt;/sup&gt;</td>
<td>Central</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
<td>Stud gauges&lt;sup&gt;3&lt;/sup&gt;</td>
<td></td>
</tr>
<tr>
<td>SWD3</td>
<td>400</td>
<td>Singles</td>
<td>0.75&lt;sup&gt;1&lt;/sup&gt;</td>
<td>Central</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Stud and waveform gauges&lt;sup&gt;1,4&lt;/sup&gt;</td>
<td></td>
</tr>
<tr>
<td>SWD4</td>
<td>1200</td>
<td>Singles</td>
<td>0.75&lt;sup&gt;1&lt;/sup&gt;</td>
<td>Central</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Stud-, waveform gauges&lt;sup&gt;1,4&lt;/sup&gt;</td>
<td></td>
</tr>
<tr>
<td>SWD5</td>
<td>600</td>
<td>Pairs</td>
<td>0.75&lt;sup&gt;1&lt;/sup&gt;</td>
<td>Central @ 80crs</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SWD6</td>
<td>1800</td>
<td>Pairs</td>
<td>0.75&lt;sup&gt;1&lt;/sup&gt;</td>
<td>Central @ 80crs</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SWD7</td>
<td>600</td>
<td>Pairs</td>
<td>0.75&lt;sup&gt;1&lt;/sup&gt;</td>
<td>Central @ 80crs</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Waveform gauges&lt;sup&gt;4&lt;/sup&gt;</td>
<td></td>
</tr>
<tr>
<td>SWD8</td>
<td>1800</td>
<td>Pairs</td>
<td>0.75&lt;sup&gt;1&lt;/sup&gt;</td>
<td>Central @ 80crs</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Waveform gauges&lt;sup&gt;4&lt;/sup&gt;</td>
<td></td>
</tr>
<tr>
<td>SWD9</td>
<td>600</td>
<td>Pairs</td>
<td>Solid slab&lt;sup&gt;2&lt;/sup&gt;</td>
<td>Central @ 80crs</td>
<td>N/A</td>
<td>No</td>
<td>No</td>
<td>Stud gauges&lt;sup&gt;3&lt;/sup&gt;</td>
<td></td>
</tr>
<tr>
<td>SWD10</td>
<td>600</td>
<td>Pairs</td>
<td>Solid slab&lt;sup&gt;2&lt;/sup&gt;</td>
<td>Central @ 80crs</td>
<td>N/A</td>
<td>No</td>
<td>No</td>
<td>Stud gauges&lt;sup&gt;3&lt;/sup&gt;, Teflon layer&lt;sup&gt;5&lt;/sup&gt;</td>
<td></td>
</tr>
</tbody>
</table>

1: steel decking anchored at its transverse edges  
2: reinforced with welded top and bottom mesh of 6N12 long. and 6N10 transv. reinforcement bars  
3: stud gauges placed in shank of front and rear studs  
4: waveform gauges located on the vertical sections of the longitudinal bars positioned in the rear of the concrete ribs.  
5: layer positioned between steel-concrete interface

**Figure 4-12:** Position of steel strain gauges on waveform element

### 4.2.2.6 CTR-RIB5 series

The final test series consisted mainly of specimens accompanying the composite beam tests described in Chapter 7 (SDM1-SDM4). In these specimens, the steel
Push-out tests – secondary beam application

Decking was not anchored and the stud arrangement was diagonal to accommodate the lap joints and stiffeners which exist in the centre of the rib when this type of decking is used in practice. The specimens accompanying the edge beam tests were of an eccentric layout in the transverse direction with a 300mm outstand measured from the centreline of the specimen to the closest edge (SDM3 and SDM4). The accompanying specimens were reinforced similar to the composite beam tests (reinforcement mesh SL72 as top layer and, if no waveform reinforcement was used, also as bottom layer placed on top of the steel decking). However, the top layer of mesh reinforcement was strengthened by welding additional longitudinal bars to it in order to prevent a ‘back-breaking’ failure, as described in Chapter 3.1.2.2, which is believed to not occur in the much stiffer slabs of a composite beam. All accompanying specimens were also equipped with stud strain gauges. The variables investigated in these tests were the concrete slab outstand and the use of shear connection reinforcing elements. The other two tests (SDM5 and SDM6) were designed to provide information on the shear connection behaviour if the waveform reinforcement element is used on its own (without the spiral enhancing device) in both, narrow and wide concrete slabs.

Table 4-10: Test program CTR-RIB5 series

<table>
<thead>
<tr>
<th>Code</th>
<th>Slab width (mm)</th>
<th>Studs</th>
<th>Sheet thickness (mm)</th>
<th>Stud placement</th>
<th>Bottom reinforcement</th>
<th>Waveform element</th>
<th>Spiral device</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>SDM1</td>
<td>1800</td>
<td>Pairs</td>
<td>0.75</td>
<td>Diagonal @ 80crs³</td>
<td>Yes</td>
<td>No</td>
<td>No</td>
<td>Stud gauges⁴</td>
</tr>
<tr>
<td>SDM2</td>
<td>1800</td>
<td>Pairs</td>
<td>0.75</td>
<td>Diagonal @ 80crs³</td>
<td>No</td>
<td>Yes</td>
<td>Yes</td>
<td>Stud gauges⁴</td>
</tr>
<tr>
<td>SDM3</td>
<td>1200¹</td>
<td>Pairs</td>
<td>0.75</td>
<td>Diagonal @ 80crs³</td>
<td>Yes</td>
<td>No</td>
<td>No</td>
<td>Stud gauges⁴</td>
</tr>
<tr>
<td>SDM4</td>
<td>1200¹</td>
<td>Pairs</td>
<td>0.75</td>
<td>Diagonal @ 80crs³</td>
<td>No</td>
<td>Yes</td>
<td>Yes</td>
<td>Stud gauges⁴</td>
</tr>
<tr>
<td>SDM5</td>
<td>600</td>
<td>Pairs</td>
<td>0.75²</td>
<td>Central @ 80crs</td>
<td>No</td>
<td>Yes</td>
<td>No</td>
<td></td>
</tr>
<tr>
<td>SDM6</td>
<td>1800</td>
<td>Pairs</td>
<td>0.75²</td>
<td>Central @ 80crs</td>
<td>No</td>
<td>Yes</td>
<td>No</td>
<td></td>
</tr>
</tbody>
</table>

¹: eccentric layout with edge distance of 300mm from centreline at closest edge
²: steel decking anchored at its transverse edges
³: studs placed 35mm in designated longitudinal direction away from lap joint or stiffener
⁴: stud gauges placed in shank of rear studs
4.3 Test results and observations

4.3.1 Shear connection in KF70® geometry

4.3.1.1 General

The material properties of the steel decking and shear connectors were determined with the help of tensile tests which were performed in accordance with AS1391 (Standards Australia 2005) using three or more test pieces. They are summarized in Table 4-11.

<table>
<thead>
<tr>
<th>Material</th>
<th>Yield strength $f_y$ (MPa)</th>
<th>Ultimate strength $f_u$ (MPa)</th>
<th>Elastic Modulus $E$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>KF70® steel decking, t=0.60mm</td>
<td>675</td>
<td>680</td>
<td>217000</td>
</tr>
<tr>
<td>KF70® steel decking, t=0.75mm</td>
<td>629</td>
<td>632</td>
<td>200000</td>
</tr>
<tr>
<td>KF70® steel decking, t=1.00 mm</td>
<td>615</td>
<td>620</td>
<td>201000</td>
</tr>
<tr>
<td>Studs, 100mm high</td>
<td>-</td>
<td>460</td>
<td>-</td>
</tr>
<tr>
<td>Studs, 125mm high</td>
<td>-</td>
<td>417</td>
<td>-</td>
</tr>
</tbody>
</table>

Once a specimen reached its designated concrete target strength, its formwork was removed and the steel flange plate of the specimen was welded to a RHS tube segment and attached to the push-out test rig as described in Chapter 3.2.2. For the secondary specimens, the vertical load was usually applied over the full specimen width (without recess) and the height of the solid cover slab (see also Figure 3-12).

The testing was generally displacement controlled, while the speed varied during testing. Each specimen was cycled at least three times between 10kN and a proof load of about 40% of the expected maximum shear strength. Each cycle lasted at least four minutes, representative of relatively slow loading of a floor in a building structure. After the last cycle, the slip was applied continuously until stud failure occurred or, if not, until the load had dropped a significant amount below the maximum.
The maximum shear strength measured before the load started to drop for the first time, the measured concrete strengths and failure modes experienced are summarized for each test series in Table 4-12 to Table 4-16. Furthermore, as an indication of the ductility of the specimen, its strength at 6 mm slip is also stated in these tables. (Eurocode 4 (CEN 2004) requires the characteristic slip capacity of a shear connection to be at least 6 mm to be classified as ductile). The load-slip curves for the tests are given in Figure 4-13 to Figure 4-19.

4.3.1.2 SRG-P01 series

<table>
<thead>
<tr>
<th>Code</th>
<th>Stud placement</th>
<th>Waveform element</th>
<th>Ring device</th>
<th>Measured concrete strength (MPa)</th>
<th>Failure mode</th>
<th>Max stud strength $P_{max}$ (kN)</th>
<th>Stud strength at 6mm slip $P_{6mm}$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S3</td>
<td>Central/ favourable</td>
<td>Yes</td>
<td>Yes, 40mm high</td>
<td>23.5</td>
<td>RPT/SS</td>
<td>90</td>
<td>86</td>
</tr>
<tr>
<td>S4</td>
<td>Central/ unfavourable</td>
<td>Yes</td>
<td>Yes, 70mm high</td>
<td>23.1</td>
<td>RPT/SS/SP</td>
<td>80</td>
<td>69</td>
</tr>
</tbody>
</table>

1: RPT: Rib punch-through, SS: Stud shearing, RS: Rib shearing, SP: Stud pull-out

Observations:

- The application of the ring enhancing device and waveform reinforcement element did not prevent the formation of rib punch-through failures completely (if the studs are placed in favourable or central position), but their combination did result in ductile behaviour and a full utilization of the stud connector strength in a single stud application (Stud shearing failure in S3).
- Where a higher ring enhancing device was used, the stud pulled out of the cover slab (S4).
4.3.1.3 SRG-P02 series

Table 4-13: Test results SRG-P02 series

<table>
<thead>
<tr>
<th>Code</th>
<th>Stud placement</th>
<th>Transv. stud spacing (mm)</th>
<th>Ring device</th>
<th>Measured concrete strength (MPa)</th>
<th>Failure mode(\dagger)</th>
<th>Max. stud strength (P_{\text{max}}) (kN)</th>
<th>Stud strength at 6mm slip (P_{\text{6mm}}) (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>Central/diagonal</td>
<td>80</td>
<td>Yes</td>
<td>32.9</td>
<td>RPT/SP</td>
<td>75</td>
<td>61</td>
</tr>
<tr>
<td>S2</td>
<td>Central/diagonal</td>
<td>80</td>
<td>Yes</td>
<td>32.9</td>
<td>RPT/SP</td>
<td>70</td>
<td>0</td>
</tr>
<tr>
<td>S3</td>
<td>Central/diagonal</td>
<td>120</td>
<td>No</td>
<td>33.9</td>
<td>RPT/SP</td>
<td>75</td>
<td>42</td>
</tr>
<tr>
<td>S4</td>
<td>Central/diagonal</td>
<td>120</td>
<td>Yes</td>
<td>33.9</td>
<td>RPT/SP</td>
<td>70</td>
<td>59</td>
</tr>
<tr>
<td>S5</td>
<td>Central/favourable</td>
<td>80</td>
<td>No</td>
<td>36.2</td>
<td>SP</td>
<td>68</td>
<td>0</td>
</tr>
<tr>
<td>S6</td>
<td>Central/favourable</td>
<td>80</td>
<td>Yes</td>
<td>37.9</td>
<td>RPT/SP</td>
<td>83</td>
<td>77</td>
</tr>
</tbody>
</table>

\(\dagger\): RPT: Rib punch-through, SS: Stud shearing, RS: Rib shearing, SP: Stud pull-out

Observations:

- Conventionally reinforced specimens with stud pairs experienced very brittle behaviours (S3 and S5).
- The specimens generally experienced a combined rib punch-through and stud pull-out failure in both ribs. Only specimen S5 experienced a pure stud pull-out failure.
The use of the ring enhancing device on its own improved the strength only slightly but did not alter the failure mode (S1, S2, S4 and S6).

The transverse stud spacing, the positioning of the studs and the concrete strength did not seem to have a major influence on the shear connection behaviour.

The external horizontal restraint at the base of the concrete slab resulted in an increase in strength and ductility but did not alter the failure mode (S6).

![Figure 4-14: Load-slip behaviour of SRG-P02 specimens](image)

4.3.1.4 SRG-P03 series

Observations:

- Large variation in the shear connection strength of similar conventionally reinforced specimens appeared (SR2 and SR3).
- The single stud shear connections in conventionally reinforced specimens (SR1-SR3) generally experienced sufficient ductility but mostly significantly lower strengths than the companion solid slab specimen (SR12).
- The use of the ring enhancing device in single stud specimens without the use of any further reinforcing components led to stud pull-out failure (SR4).
- All specimens with stud pair arrangements did not reach the solid slab shear connection strength of specimen SR12.
The use of waveform reinforcement elements including transverse cross-wires in stud pair specimens led to much improved shear connection behaviour. The cross-wires used in conjunction with the waveform reinforcement element did successfully suppress stud pull-out failures (SR5-SR6).

The addition of the ring enhancing device increased the maximum strength slightly, but caused brittle rib-shearing failure at the edges of the specimen (SR8-SR10). This failure might not have been experienced in specimens with wider concrete slabs.

The increased stud height in a conventionally reinforced specimen including stud pairs resulted in a more ductile behaviour but not in an increase in strength (Specimen SR13 still experienced punch-through failure, but no stud pull-out failure).

<table>
<thead>
<tr>
<th>Code</th>
<th>Studs (height in mm)</th>
<th>Cross-wire in ribs</th>
<th>Waveform element</th>
<th>Ring device</th>
<th>Measured concrete strength (MPa)</th>
<th>Failure mode</th>
<th>Max. stud strength $P_{\text{max}}$ (kN)</th>
<th>Stud strength at 6mm slip $P_{\text{slip}}$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SR1</td>
<td>Singles (100)</td>
<td>No</td>
<td>No</td>
<td>No</td>
<td>28.4</td>
<td>RPT/SS</td>
<td>92</td>
<td>89</td>
</tr>
<tr>
<td>SR2</td>
<td>Singles (100)</td>
<td>No</td>
<td>No</td>
<td>No</td>
<td>41.0</td>
<td>RPT/SS</td>
<td>89</td>
<td>85</td>
</tr>
<tr>
<td>SR3</td>
<td>Singles (100)</td>
<td>No</td>
<td>No</td>
<td>No</td>
<td>41.0</td>
<td>RPT/SS</td>
<td>119</td>
<td>114</td>
</tr>
<tr>
<td>SR4</td>
<td>Singles (100)</td>
<td>No</td>
<td>No</td>
<td>Yes</td>
<td>32.7</td>
<td>RPT/SP</td>
<td>120</td>
<td>111</td>
</tr>
<tr>
<td>SR5</td>
<td>Pairs (100)</td>
<td>Yes</td>
<td>Yes</td>
<td>No</td>
<td>38.2</td>
<td>RPT/RS</td>
<td>91</td>
<td>85</td>
</tr>
<tr>
<td>SR6</td>
<td>Pairs (100)</td>
<td>Yes</td>
<td>Yes</td>
<td>No</td>
<td>30.8</td>
<td>RPT</td>
<td>87</td>
<td>84</td>
</tr>
<tr>
<td>SR7</td>
<td>Pairs (100)</td>
<td>No</td>
<td>Yes</td>
<td>No</td>
<td>30.8</td>
<td>RPT/RS/SP</td>
<td>77</td>
<td>57</td>
</tr>
<tr>
<td>SR8</td>
<td>Pairs (100)</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>39.2</td>
<td>RPT/RS</td>
<td>97</td>
<td>80</td>
</tr>
<tr>
<td>SR9</td>
<td>Pairs (100)</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>34.4</td>
<td>RPT/RS</td>
<td>96</td>
<td>0</td>
</tr>
<tr>
<td>SR10</td>
<td>Pairs (100)</td>
<td>No</td>
<td>Yes</td>
<td>Yes</td>
<td>34.4</td>
<td>RPT/RS/SP</td>
<td>86</td>
<td>73</td>
</tr>
<tr>
<td>SR12</td>
<td>Pairs (100)</td>
<td>N/A</td>
<td>No</td>
<td>No</td>
<td>32.7</td>
<td>SS</td>
<td>118</td>
<td>110</td>
</tr>
<tr>
<td>SR13</td>
<td>Pairs (130)</td>
<td>No</td>
<td>No</td>
<td>No</td>
<td>38.2</td>
<td>RPT/SS</td>
<td>70</td>
<td>67</td>
</tr>
</tbody>
</table>

1: RPT: Rib punch-through, SS: Stud shearing, RS: Rib shearing, SP: Stud pull-out
Figure 4-15: Load-slip behaviour of SRG-P03 single stud specimens

Figure 4-16: Load-slip behaviour of SRG-P03 stud pair specimens without ring enhancing device
4.3.1.5 OSR-DM01 series

Observations:

- The behaviour of single stud shear connections in conventional reinforced specimens was very dependent on concrete strength and was of a brittle nature (S10-S12).
- The cross-wire in the concrete rib (S07-S09) increased the strength of single stud shear connections.
- Stud pair specimens including the waveform reinforcement element generally experienced a sufficiently ductile behaviour (S01-S03).
- The shear connection behaviour seemed to be less influenced by the concrete strength if the waveform reinforcement element was provided.
- There were large variations in the behaviour of specimens incorporating pairs of studs where the cross-wire was provided in the concrete ribs (S01-S03).
- Numerous specimens with stud pairs experienced failures at the edges of the slab. Wider specimens might have provided better results.
### Table 4-15: Test results OSR-DM01 series

<table>
<thead>
<tr>
<th>Code</th>
<th>Studs</th>
<th>Cross-wire in ribs</th>
<th>Wave-form element</th>
<th>Measured concrete strength (MPa)</th>
<th>Failure mode</th>
<th>Max. stud strength $P_{\text{max}}$ (kN)</th>
<th>Stud strength at 6mm slip $P_{\text{6mm}}$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S01</td>
<td>Pairs</td>
<td>Yes</td>
<td>Yes</td>
<td>26.6</td>
<td>RPT/RS²</td>
<td>80</td>
<td>80</td>
</tr>
<tr>
<td>S02</td>
<td>Pairs</td>
<td>Yes</td>
<td>Yes</td>
<td>32.8</td>
<td>RPT²</td>
<td>93</td>
<td>88</td>
</tr>
<tr>
<td>S03</td>
<td>Pairs</td>
<td>Yes</td>
<td>Yes</td>
<td>32.1</td>
<td>RPT/RS</td>
<td>81</td>
<td>81</td>
</tr>
<tr>
<td>S04</td>
<td>Pairs</td>
<td>Yes</td>
<td>No</td>
<td>26.6</td>
<td>RPT/RS²</td>
<td>72</td>
<td>63</td>
</tr>
<tr>
<td>S05</td>
<td>Pairs</td>
<td>Yes</td>
<td>No</td>
<td>31.3</td>
<td>RPT/SP</td>
<td>78</td>
<td>53</td>
</tr>
<tr>
<td>S06</td>
<td>Pairs</td>
<td>Yes</td>
<td>No</td>
<td>32.3</td>
<td>RPT</td>
<td>67</td>
<td>52</td>
</tr>
<tr>
<td>S07</td>
<td>Singles</td>
<td>Yes</td>
<td>No</td>
<td>30.2</td>
<td>RPT/SP³</td>
<td>129</td>
<td>92</td>
</tr>
<tr>
<td>S08</td>
<td>Singles</td>
<td>Yes</td>
<td>No</td>
<td>32.2</td>
<td>RPT</td>
<td>131</td>
<td>128</td>
</tr>
<tr>
<td>S09</td>
<td>Singles</td>
<td>Yes</td>
<td>No</td>
<td>35.3</td>
<td>RPT</td>
<td>139</td>
<td>129</td>
</tr>
<tr>
<td>S10</td>
<td>Singles</td>
<td>No</td>
<td>No</td>
<td>29.6</td>
<td>RPT/SP</td>
<td>92</td>
<td>74</td>
</tr>
<tr>
<td>S11</td>
<td>Singles</td>
<td>No</td>
<td>No</td>
<td>31.9</td>
<td>RPT/SP</td>
<td>100</td>
<td>97</td>
</tr>
<tr>
<td>S12</td>
<td>Singles</td>
<td>No</td>
<td>No</td>
<td>39.8</td>
<td>RPT/SP</td>
<td>117</td>
<td>103</td>
</tr>
</tbody>
</table>

1: RPT: Rib punch-through, SS: Stud shear, RS: Rib shearing, SP: Stud pull-out
2: initial splitting of concrete slab under loading beam (see also Chapter 3.2.3), specimen was cut and test restarted
3: test aborted, insufficient welds between steel plate and RHS section

![Figure 4-18: Load-slip behaviour of OSR-DM01 stud pair specimens](image)
### OBD-HS01 series

#### Table 4-16: Test results OBD-HS01 series

<table>
<thead>
<tr>
<th>Code</th>
<th>Studs</th>
<th>Stud placement</th>
<th>Ring device</th>
<th>Measured concrete strength (MPa)</th>
<th>Failure mode(^1)</th>
<th>Max. stud strength (P_{\text{max}}) (kN)</th>
<th>Stud strength at 6mm slip (P_{6\text{mm}}) (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S01</td>
<td>Singles</td>
<td>Central/ favourable</td>
<td>No</td>
<td>48.2</td>
<td>RPT</td>
<td>111</td>
<td>100</td>
</tr>
<tr>
<td>S02</td>
<td>Singles</td>
<td>Central/ unfavourable</td>
<td>No</td>
<td>48.5</td>
<td>RPT(^2)</td>
<td>108</td>
<td>93</td>
</tr>
<tr>
<td>S03</td>
<td>Singles</td>
<td>Central/ favourable</td>
<td>Yes</td>
<td>48.5</td>
<td>RPT(^2)</td>
<td>114</td>
<td>107</td>
</tr>
<tr>
<td>S04</td>
<td>Singles</td>
<td>Central/ unfavourable</td>
<td>Yes</td>
<td>49.1</td>
<td>RPT/RS</td>
<td>132</td>
<td>124</td>
</tr>
</tbody>
</table>

\(^1\): RPT: Rib punch-through, SS: Stud shear, RS: Rib shearing, SP: Stud pull-out  
\(^2\): initial splitting of concrete slab under loading beam (see also Chapter 3.2.3), specimen was cut and test restarted at 5.5mm slip

Observations:

- Comparing specimen S01 with S02, the position of the shear connector in the concrete rib did not seem to have a significant impact on the shear connection strength. Both conventional reinforced specimens (with comparable larger embedment depths of the studs) still experienced rib punch-through failures resulting in a significant drop in load, but no stud pull-out failures.
The ring enhancing device used on its own increased the shear connection strength but could still not provide sufficient ductility even if higher studs were used (S03 and S04).

Wider specimens might have prevented the rib shearing failure which was experienced for specimen S04.

A change in test-setup was anticipated as large rotations of the loading beam were observed in all test specimens of this series (see also Chapter 3.2.3).

![Figure 4-20: Load-slip behaviour of OBD-HS01 specimens](image)

**Figure 4-20:** Load-slip behaviour of OBD-HS01 specimens

### 4.3.2 Shear connection in W-Dek® geometry

#### 4.3.2.1 General

The properties of the steel decking and the headed studs were measured in separate tensile tests in accordance with AS1391 (Standards Australia 2005). The material properties are summarized in Table 4-17.
Table 4-17: Steel decking and stud material properties of W-Dek® tests

<table>
<thead>
<tr>
<th>Material</th>
<th>Yield strength $f_y$ (MPa)</th>
<th>Ultimate strength $f_u$ (MPa)</th>
<th>Elastic Modulus $E$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>W-Dek® steel decking, t=0.75mm</td>
<td>626</td>
<td>629</td>
<td>200000</td>
</tr>
<tr>
<td>W-Dek® steel decking, t=1.00 mm</td>
<td>579</td>
<td>587</td>
<td>184000</td>
</tr>
<tr>
<td>Studs, 127mm high (Specimens SDM1-SDM4)</td>
<td>-</td>
<td>550</td>
<td>-</td>
</tr>
<tr>
<td>Studs, 127mm high (all other specimens)</td>
<td>-</td>
<td>509</td>
<td>-</td>
</tr>
</tbody>
</table>

Each specimen was stripped of its formwork once the desired concrete strength was reached. It was then welded to a steel connection plate and attached to the horizontal push-out test rig described in Chapter 3.3.2. The instrumentation of the specimens was carried out in a similar fashion to previous tests where the slips of the shear connections were measured with the use of steel rods embedded into the concrete slabs through holes cut into the steel decking in the vicinity of the studs. The attachment of the linear potentiometers to the sliding beam via magnetic bases is shown indicatively in Figure 4-21. Five linear potentiometers were positioned horizontally to measure the longitudinal slip between the slab and the steel beam (two adjacent to both sides of the front stud, two adjacent to the back stud, and one at the end of the specimen). Four linear potentiometers were positioned vertically to measure the separation between the slab and the steel beam, two at the front rib and two at the back rib of the specimen. Vertical potentiometers were also used to measure the vertical movement of the centre of the flanges of the transverse restraining beam. The tests were started after the mortar, which was applied between concrete slab and restraining beam, had set.

The test procedure was the same as for the KF70® specimens: After cycling the specimens three times at a rate of at least 4 minutes per cycle, the slip was applied continuously until stud failure occurred or, if not, until the load had dropped a significant amount below the maximum. Owing to very large deformation in some of the tests, the actuator ran out of stroke before complete failures were observed in which case the tests were terminated.
The shear connection strength at maximum load and 6mm slip, the measured concrete strength and the failure modes experienced by the various test specimens are summarized in Table 4-18 to Table 4-22. The load-slip curves for each test specimen are also given in Figure 4-22 to Figure 4-28.

### 4.3.2.2 CTR-RIB1 series

Observations:

- All secondary beam specimens tested (SW1-SW7) experienced the same failure mode: rib punch-through failure.
- The specimens with steel decking (SW1 and SW2) reached a maximum of about 2/3 of the solid slab strength (SW8).
- No visible influence of the reinforcement bottom layer on the shear connection behaviour could be observed. (SW1-SW4)
- If steel decking was omitted, further reduction of the stud strengths and sharp drops in loads were experienced which resulted in more brittle behaviours of the shear connections (SW3-SW7).
- After reaching the second maximum at very large slips, some of the shear connectors fractured while other tests were stopped prior that point.
- The influence of the specimen width on the stud strength was very small and did not alter the failure mode (SW3, SW6-SW7).
- An increase in concrete strength also increased the shear connection strength for the single stud secondary beam application (SW5).

### Table 4-18: Test results CTR-RIB1 series

<table>
<thead>
<tr>
<th>Code</th>
<th>Width (mm)</th>
<th>Studs</th>
<th>Sheet</th>
<th>Bottom reinforcement</th>
<th>Measured concrete strength (MPa)</th>
<th>Failure mode $P_{\text{max}}$ (kN)</th>
<th>Stud strength at 6mm slip $P_{\text{6mm}}$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SW1</td>
<td>1200</td>
<td>Singles</td>
<td>0.75</td>
<td>Yes</td>
<td>29.6</td>
<td>RPT</td>
<td>100.1</td>
</tr>
<tr>
<td>SW2</td>
<td>1200</td>
<td>Singles</td>
<td>0.75</td>
<td>No</td>
<td>30.7</td>
<td>RPT</td>
<td>99.7</td>
</tr>
<tr>
<td>SW3</td>
<td>1200</td>
<td>Singles</td>
<td>No sheet</td>
<td>Yes</td>
<td>29.8</td>
<td>RPT</td>
<td>84.1</td>
</tr>
<tr>
<td>SW4</td>
<td>1200</td>
<td>Singles</td>
<td>No sheet</td>
<td>No</td>
<td>30.9</td>
<td>RPT</td>
<td>77.6</td>
</tr>
<tr>
<td>SW5</td>
<td>900</td>
<td>Singles</td>
<td>No sheet</td>
<td>Yes</td>
<td>41.4</td>
<td>RPT</td>
<td>95.0</td>
</tr>
<tr>
<td>SW6</td>
<td>600</td>
<td>Singles</td>
<td>No sheet</td>
<td>Yes</td>
<td>34.3</td>
<td>RPT</td>
<td>76.7</td>
</tr>
<tr>
<td>SW7</td>
<td>400</td>
<td>Singles</td>
<td>No sheet</td>
<td>Yes</td>
<td>33.8</td>
<td>RPT</td>
<td>74.2</td>
</tr>
<tr>
<td>SW8</td>
<td>600</td>
<td>Singles</td>
<td>Solid slab</td>
<td>N/A</td>
<td>33.2</td>
<td>SS</td>
<td>147.4</td>
</tr>
</tbody>
</table>

$1$: RPT: Rib punch-through, SS: Stud shearing, RS: Rib shearing, SP: Stud pull-out

**Figure 4-22: Load-slip behaviour of CTR-RIB1 specimens**
4.3.2.3 CTR-RIB2 series

<table>
<thead>
<tr>
<th>Code</th>
<th>Width (mm)</th>
<th>Studs</th>
<th>Sheet</th>
<th>Measured concrete strength (MPa)</th>
<th>Failure mode</th>
<th>Max. stud strength $P_{\text{max}}$ (kN)</th>
<th>Stud strength at 6mm slip $P_{\text{6mm}}$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SWP1</td>
<td>900</td>
<td>Pairs</td>
<td>No sheet</td>
<td>28.3</td>
<td>RPT/RS/SP</td>
<td>51.6</td>
<td>37.5</td>
</tr>
<tr>
<td>SWP2</td>
<td>600</td>
<td>Pairs</td>
<td>No sheet</td>
<td>28.5</td>
<td>RPT/RS</td>
<td>51.6</td>
<td>32.3</td>
</tr>
<tr>
<td>SWP3</td>
<td>400</td>
<td>Pairs</td>
<td>No sheet</td>
<td>29.1</td>
<td>RPT/RS</td>
<td>47.4</td>
<td>31</td>
</tr>
</tbody>
</table>

1: RPT: Rib punch-through, SS: Stud shearing, RS: Rib shearing, SP: Stud pull-out

Observations:

- A very low shear connector strength and non-ductile behaviour was characteristic for all stud pair specimens.
- All specimens initially experienced the formation of rib punch-through cracks but eventually the load started to drop when horizontal rib shearing cracks became visible at the edges of the concrete slabs.
- The rear rib of the wider specimen SWP1 experienced stud pull-out cracks rather than rib shearing cracks on one edge. Hence, the failure mode might be altered if wider specimens were tested.

![Figure 4-23: Load-slip behaviour of CTR-RIB2 specimens](image-url)
### 4.3.2.4 CTR-RIB3 series

#### Table 4-20: Test results CTR-RIB3 series

<table>
<thead>
<tr>
<th>Code</th>
<th>Width (mm)</th>
<th>Studs</th>
<th>Sheet</th>
<th>Spiral device</th>
<th>Measured concrete strength (MPa)</th>
<th>Failure mode†</th>
<th>Max. stud strength $P_{\text{max}}$ (kN)</th>
<th>Stud strength at 6mm slip $P_{6\text{mm}}$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SWR1</td>
<td>1200</td>
<td>Pairs</td>
<td>No sheet</td>
<td>No</td>
<td>31.9</td>
<td>RPT/ (SP)</td>
<td>56.0</td>
<td>34.6</td>
</tr>
<tr>
<td>SWR2</td>
<td>1800</td>
<td>Pairs</td>
<td>No sheet</td>
<td>No</td>
<td>29.3</td>
<td>RPT</td>
<td>64.2</td>
<td>49.3</td>
</tr>
<tr>
<td>SWR3</td>
<td>600</td>
<td>Pairs</td>
<td>No sheet</td>
<td>Yes</td>
<td>31.7</td>
<td>RPT/ RS</td>
<td>64.5</td>
<td>35.7</td>
</tr>
<tr>
<td>SWR4</td>
<td>1200</td>
<td>Pairs</td>
<td>No sheet</td>
<td>Yes</td>
<td>32.3</td>
<td>RPT/ SP/RS</td>
<td>67.2</td>
<td>48.2</td>
</tr>
<tr>
<td>SWR5</td>
<td>1800</td>
<td>Pairs</td>
<td>No sheet</td>
<td>Yes</td>
<td>30.1</td>
<td>RPT/ SP</td>
<td>77.8</td>
<td>57.1</td>
</tr>
<tr>
<td>SWR6</td>
<td>600</td>
<td>Pairs</td>
<td>1.00</td>
<td>No</td>
<td>32.9</td>
<td>RPT/ RS</td>
<td>69.7</td>
<td>54.9</td>
</tr>
<tr>
<td>SWR7</td>
<td>1200</td>
<td>Pairs</td>
<td>1.00</td>
<td>No</td>
<td>32.6</td>
<td>RPT</td>
<td>75.5</td>
<td>70.7</td>
</tr>
<tr>
<td>SWR8</td>
<td>400</td>
<td>Singles</td>
<td>No sheet</td>
<td>Yes</td>
<td>33.4</td>
<td>RPT/ RS</td>
<td>82.1</td>
<td>54.2</td>
</tr>
<tr>
<td>SWR9</td>
<td>1200</td>
<td>Singles</td>
<td>No sheet</td>
<td>Yes</td>
<td>32.0</td>
<td>RPT/ SP</td>
<td>107.8</td>
<td>93.9</td>
</tr>
</tbody>
</table>

†: RPT: Rib punch-through, SS: Stud shearing, RS: Rib shearing, SP: Stud pull-out

Observations:

- The wider, conventionally reinforced stud pair specimens (SWR1 and SWR2) experienced a significant drop in load after rib punch-through failure occurred but neither consecutive rib shearing nor stud pull-out failures were observed. However, in specimen SWR1, stud pull-out cracks had started to form in one rib prior to when the rib punch-through failure occurred but had not fully developed as the load started to drop.

- The use of steel decking in stud pair specimens (SWR6 and SWR7) did increase the shear connection strength but it did not alter its behaviour. In the narrow specimen, the decking could not provide sufficient resistance against rib shearing failure (SWR6) while in the wider specimen, it was able to reduce the drop in load after rib punch-through failure occurred (SWR7).

- The use of the spiral enhancing device increased the shear connection strengths but could not completely suppress the formation of rib punch through cracks. In addition, these specimens experienced brittle stud pull-out failures in the wider concrete slabs, or rib shearing failures in the narrow
slabs for both single stud and stud pair applications (SWR3-SWR5 and SWR8-SWR9).

**Figure 4-24:** Load-slip behaviour of conventionally reinforced CTR-RIB3 specimens

**Figure 4-25:** Load-slip behaviour of CTR-RIB3 specimens including the spiral enhancing device
4.3.2.5 CTR-RIB4 series

Table 4-21: Test results CTR-RIB4 series

<table>
<thead>
<tr>
<th>Code</th>
<th>Width (mm)</th>
<th>Studs</th>
<th>Sheet (mm)</th>
<th>WR$^1$</th>
<th>SD$^2$</th>
<th>Measured concrete strength (MPa)</th>
<th>Failure mode$^3$</th>
<th>Max. stud strength $P_{\text{max}}$ (kN)</th>
<th>Stud strength at 6mm slip $P_{6\text{mm}}$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SWD1</td>
<td>400</td>
<td>Singles</td>
<td>0.75</td>
<td>No</td>
<td>Yes</td>
<td>33.9</td>
<td>RPT/RS</td>
<td>101.1</td>
<td>80.9</td>
</tr>
<tr>
<td>SWD2</td>
<td>1200</td>
<td>Singles</td>
<td>0.75</td>
<td>No</td>
<td>Yes</td>
<td>33.2</td>
<td>RPT/SP</td>
<td>114.6</td>
<td>107.6</td>
</tr>
<tr>
<td>SWD3</td>
<td>400</td>
<td>Singles</td>
<td>0.75</td>
<td>Yes</td>
<td>Yes</td>
<td>33.6</td>
<td>RPT/SS</td>
<td>121.1</td>
<td>113.6</td>
</tr>
<tr>
<td>SWD4</td>
<td>1200</td>
<td>Singles</td>
<td>0.75</td>
<td>Yes</td>
<td>Yes</td>
<td>32.8</td>
<td>RPT/SS</td>
<td>139.5</td>
<td>129.1</td>
</tr>
<tr>
<td>SWD5</td>
<td>600</td>
<td>Pairs</td>
<td>0.75</td>
<td>No</td>
<td>Yes</td>
<td>30.9</td>
<td>RPT/RS</td>
<td>64.4</td>
<td>50.2</td>
</tr>
<tr>
<td>SWD6</td>
<td>1800</td>
<td>Pairs</td>
<td>0.75</td>
<td>No</td>
<td>Yes</td>
<td>30.0</td>
<td>RPT/SP</td>
<td>73.8</td>
<td>62.5</td>
</tr>
<tr>
<td>SWD7</td>
<td>600</td>
<td>Pairs</td>
<td>0.75</td>
<td>Yes</td>
<td>Yes</td>
<td>30.5</td>
<td>RPT</td>
<td>86.1</td>
<td>80.2</td>
</tr>
<tr>
<td>SWD8</td>
<td>1800</td>
<td>Pairs</td>
<td>0.75</td>
<td>Yes</td>
<td>Yes</td>
<td>29.8</td>
<td>RPT</td>
<td>96.6</td>
<td>94.8</td>
</tr>
<tr>
<td>SWD9</td>
<td>600</td>
<td>Pairs</td>
<td>Solid slab</td>
<td>No</td>
<td>No</td>
<td>31.4</td>
<td>SS</td>
<td>107.75</td>
<td>126.1</td>
</tr>
<tr>
<td>SWD10</td>
<td>600</td>
<td>Pairs</td>
<td>Solid slab</td>
<td>No</td>
<td>No</td>
<td>31.5</td>
<td>SS</td>
<td>101.1</td>
<td>130.9</td>
</tr>
</tbody>
</table>

$^1$: WR: Waveform reinforcement element, $^2$: SD: Spiral enhancing device
$^3$: RPT: Rib punch-through, SS: Stud shearing, RS: Rib shearing, SP: Stud pull-out

Observations:

- The application of the spiral enhancing device in combination with steel decking increased the strength significantly but resulted in a brittle rib shearing failure in narrow specimens (SWD1 and SWD5) or stud pull-out failure in wide specimens (SWD2 and SWD6).
- The combination of the spiral enhancing device with the waveform reinforcement element increased the shear connection strength further and additionally provided sufficient ductility in the specimens tested (SWD3-SWD4 and SWD7-SWD8).
- For the single stud specimens including spiral enhancing devices and waveform elements, behaviours similar to a solid slab application could be observed with the governing failure being stud shearing in both, narrow and wide concrete slabs (SWD3 and SWD4). However, the strength of the narrow specimen was slightly lower.
The inclusion of a Teflon layer (SWD10), i.e. the reduction of friction at the steel-concrete interface in solid slab applications did not have any significant effects on the shear connection behaviour (Specimens SWD9-10).

**Figure 4-26:** Load-slip behaviour of CTR-RIB4 single stud specimens

**Figure 4-27:** Load-slip behaviour of CTR-RIB4 stud pair specimens
4.3.2.6 CTR-RIB5 series

Table 4-22: Test results CTR-RIB5 series

<table>
<thead>
<tr>
<th>Code</th>
<th>Width (mm)</th>
<th>Studs</th>
<th>Sheet (mm)</th>
<th>WR</th>
<th>SD</th>
<th>Measured concrete strength (MPa)</th>
<th>Failure mode</th>
<th>Max. stud strength $P_{\text{max}}$ (kN)</th>
<th>Stud strength at 6mm slip $P_{\text{6mm}}$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SDM1</td>
<td>1800</td>
<td>Pairs</td>
<td>0.75</td>
<td>No</td>
<td>No</td>
<td>31.2</td>
<td>RPT/SP</td>
<td>69.8</td>
<td>68.6</td>
</tr>
<tr>
<td>SDM2</td>
<td>1800</td>
<td>Pairs</td>
<td>0.75</td>
<td>Yes</td>
<td>Yes</td>
<td>31.3</td>
<td>RPT</td>
<td>86.4</td>
<td>82.6</td>
</tr>
<tr>
<td>SDM3</td>
<td>1200$^1$</td>
<td>Pairs</td>
<td>0.75</td>
<td>No</td>
<td>No</td>
<td>33.0</td>
<td>RPT/RS</td>
<td>65.9</td>
<td>63.6</td>
</tr>
<tr>
<td>SDM4</td>
<td>1200$^1$</td>
<td>Pairs</td>
<td>0.75</td>
<td>Yes</td>
<td>Yes</td>
<td>33.5</td>
<td>RPT/RS</td>
<td>85.7</td>
<td>82.9</td>
</tr>
<tr>
<td>SDM5</td>
<td>600</td>
<td>Pairs</td>
<td>0.75</td>
<td>Yes</td>
<td>No</td>
<td>29.9</td>
<td>RPT/RS</td>
<td>61.1</td>
<td>53.5</td>
</tr>
<tr>
<td>SDM6</td>
<td>1800</td>
<td>Pairs</td>
<td>0.75</td>
<td>Yes</td>
<td>No</td>
<td>29.6</td>
<td>RPT/SS$^4$</td>
<td>86</td>
<td>71.4</td>
</tr>
</tbody>
</table>

| 1: eccentric layout with edge distance of 300mm from centreline at closest edge | 2: WR: Waveform reinforcement element, 2: SD: Spiral enhancing device |
| 3: RPT: Rib punch-through, SS: Stud shear, RS: Rib shearing, SP: Stud pull-out |
| 4: faulty stud weld on one stud |

**Observations:**

- The conventional reinforced specimen with an eccentric layout (SDM3) displayed a ductile behaviour which is in contrast to earlier tests.
- The use of the spiral enhancing device in combination with the waveform reinforcement element increased the strength in both specimens with wide and narrow concrete slabs compared to conventionally reinforced specimens (SDM2 and SDM4). Both these specimens experienced a slightly lower strength and less ductility compared to the earlier tests of series CTR-RIB4 where the stud position in the concrete rib differed and the steel decking was anchored at its edges.
- The specimens including the waveform reinforcement component on its own displayed a very ductile behaviour in a wide concrete slab application (SDM6) and a sufficiently ductile behaviour in a narrow slab application (SDM5) where, despite the application of the component, a rib-shearing crack was observed in one rib of the specimen.
- A faulty weld in one of the studs in Specimen SDM6 became visible after failure. It is thought that this insufficient weld was responsible for the
premature stud shearing failure and the comparable low strength experienced in this specimen.

![Figure 4-28: Load-slip behaviour of CTR-RIB5 specimens](image)

### 4.4 Failure modes

#### 4.4.1 Stud shearing failure

The horizontal shear force at the steel beam - concrete slab interface creates shear stresses in the shank of the stud connectors. A stud shearing failure is experienced when these stresses exceed the shear capacity of the stud. However, the failure mode is not a pure shear failure since the base region of the shear connector deforms horizontally as the concrete crushes locally in the bearing zone of the connector while the top part of the shank is still embedded in sound concrete creating an additional bending effect in the shank of the stud. Under monotonic loading, the studs typically shear off right above their welds where there is a sudden change in their cross-sectional areas. A loud noise can be heard when the first stud fails which is accompanied by an instant sharp drop in load and successive shearing failures of the remaining studs. The horizontal shear surface which developed just above the weld can be seen in Figure 4-29 while the deformed base part of the shank of the stud
and the local crushing of the concrete in front of the shear connector are clearly visible on the underside of the failed concrete slab (Figure 4-30).

![Figure 4-29: Shearing of the studs above their weld (Specimen RIB4-SWD9)](image)

The stud shearing behaviour is the only one of the observed failure modes which displayed a behaviour representing failure of the shear connector material. This behaviour is far more ductile and reliable than the concrete-related failure modes described below. No further increase in shear connection strength above stud shearing is possible as the connector is utilized to its full strength. Hence, stud shearing should be the preferable mode of failure when designing a shear connection.

![Figure 4-30: Underside of concrete slab after failure (Specimen RIB4-SWD9)](image)

In the tests undertaken, only the solid slab specimens experienced a pure stud shearing failure without the influence of prior concrete cracking in the vicinity of the
shear connection. A stud fracture above the weld was also experienced for some of the specimens which initially displayed rib punch-through behaviour. However, in these cases, the shank of the stud deformed over a much larger region which increased the influence of bending effects and decreased the strength and stiffness of the connection. Specimens which were subsequently reinforced to minimize the impact of concrete-related failure modes partly experienced stud shearing failures displaying stiffness and deformation capacities similar to those in solid slab specimens.

### 4.4.2 Rib punch-through failure

Indications of this failure mode were observed for nearly every secondary beam test specimen. The high stresses that develop in the bearing zone of a stud connector and its proximity to a free concrete edge are thought to cause a complete wedge of concrete to break out of the rib. This is normally associated with a sudden drop-off in shear strength as the studs lose confinement at their base.

![Figure 4-31: Forming of concrete wedge](image)

The specimens tested without steel decking provided a clearer insight into the behaviour of this failure mode (Figure 4-31). Firstly, cracks developed diagonally from the base of the shear connectors towards the edge of the concrete ribs in the
direction of the longitudinal shear. Once these cracks reached the edges of the concrete rib, they further propagated vertically across the height of the concrete rib. Finally, a horizontal crack formed between the two vertical cracks across the top corner of the rib. The load started to drop rapidly when the complete concrete wedge had formed.

Once the concrete wedge had formed, the shear force was transferred into the remaining undamaged concrete, and the base region of the studs underwent significant bending deformations as their upper regions were still embedded in undamaged concrete. At this stage, the load remained constant as the shear forces were transferred into the undamaged concrete and any further increase in slip was translated into an increase in bending deformation at the stud base. After the tests, the concrete wedges could be easily removed by hand from the concrete rib (Figure 4-32). Figure 4-33 shows the unconfined base of the shear connector and its bending deformation after the concrete wedge was removed.

Where steel decking was used, the decking bulged out approaching maximum load, clearly indicating the development of similar concrete wedges. In specimens where pieces of the steel decking were removed after testing, the failure wedge which developed in front of each shear connector during testing can be clearly seen (Figure 4-34). The cracked concrete could then be easily removed by hand. Some specimens
even experienced a fracture of the bulged steel decking where the concrete wedge had literally punched through the decking (Figure 4-35).

Figure 4-34: Bulging of steel decking and failure wedges after steel decking was cut out (Specimen RIB1-SW2)

Figure 4-35: Fracture of steel decking at underside of concrete slab (Specimen RIB5-SDM6)

Provided no successive concrete-related failure mode occurred, two plastic hinges formed in the shank of the studs resulting in large slips. The large deformation of the studs caused a large increase in tensile force in their shanks, which, in some cases, resulted in a second peak shear strength to be reached with the stud shanks finally fracturing just above the welds. These fractures were results of combined shear, bending and tensile stresses in the shanks. The second peak typically appeared at
more than 20mm slip so it is normally not of practical significance. This behaviour was mostly observed for the single stud specimens tested.

A typical load versus slip curve for a specimen which experienced rib punch-through failure together with measurements of the tensile forces in the stud shanks is shown in Figure 4-36. The measurements underline the correlation between stud normal forces and a second increase in the shear connection capacity. In contrast, the formation and breakout of the concrete wedges did not seem to have much effect on the stud normal forces.

![Graph](image.png)

**Figure 4-36:** Stud shear and tensile force measurements (from strain-gauge readings) for a conventionally reinforced, single stud specimen (RIB1-SW7)

### 4.4.3 Rib splitting failure

In the test specimens where the steel decking was omitted, it was observed that a longitudinal crack had developed behind each shear connector which propagated longitudinally to the backside of the concrete rib and vertically across its height (Figure 4-37). It was also observed that most of the failed concrete wedges showed a similar vertical crack or were broken in half. The longitudinal cracks were more severe where pairs of studs were tested or where stud enhancing devices were used (Figure 4-38). In specimens with steel decking, similar cracks were observed when
the sheeting was removed after testing. These types of cracks were also recorded in earlier push-out tests on the ring enhancing device (see Chapter 2.3.4.3).

![Image](image1.png)

**Figure 4-37:** Splitting cracks in a single stud application after specimen failure

![Image](image2.png)

**Figure 4-38:** Splitting cracks in a stud pair application after specimen failure

It is thought that these cracks indicate a longitudinal splitting failure of the concrete rib since its narrow geometry can not accommodate the transverse tensile forces which are induced due to the concentrated load transfer (Figure 4-39). As there is typically no transverse reinforcement in the concrete rib to accommodate these tensile forces, the rib initially splits in the concrete bearing zone in front of the shear connector, and the longitudinal crack eventually propagates to the concrete at the opposite side of the connector. This crack propagation could also be initiated by the break-out of the concrete wedge in front of the shear connector. If the loads are further increased, the transverse cracks widen and propagate further along the height.
of the concrete rib. However, a complete failure of the shear connection does not appear as the solid cover part of the concrete slab typically restricts the transverse separation of the concrete rib, and limits the propagation of the longitudinal crack to the concrete rib – cover slab interface.

![Diagram](image.png)

**Figure 4-39:** Splitting cracks induced into a narrow concrete rib by the concentrated load transfer

Longitudinal splitting in secondary composite beam applications has not been investigated in previous works as these cracks usually remain hidden under the steel decking and their appearance does not cause a complete failure of the shear connection. The failure mode was also not stated as one of the governing failure modes in the tests described in Chapter 4.3. However, it is already well established that the appearance of unreinforced splitting cracks reduces the shear connection strengths as the triaxial compression zone which develops in front of the shear connectors is weakened (Oehlers 1989). It was also found that splitting reduces the resistance against stud pull-out failures in fastener applications (Eligehausen and Mallee 2000). For the tests described in Chapter 4.3 it is thought that this failure mode generally appeared wherever no transverse reinforcement was provided in the concrete ribs.

### 4.4.4 Rib shearing failure

Rib-shearing failure is characterized by a horizontal crack propagating from the top back corner of the concrete rib to the adjacent rib corner while locally passing over the heads of the shear connectors eventually separating the solid cover part of the slab from its ribs (Figure 4-40). This failure mode can be of a very brittle nature.
when no vertical reinforcement passes through the potential failure surface. The onset of the concrete crack at the transverse edges of the concrete rib is accompanied by a sharp drop in load. For the specimens where the steel decking was omitted, it was observed that the horizontal rib shearing cracks typically appeared simultaneously with the formation of the rib punch-through wedges.

In Figure 4-41, the readings of the vertical strain gauges embedded in the concrete slab at the interface between concrete rib and solid cover part can be correlated with the load-slip curve for a specimen that experienced rib-shearing failure. Once the load-slip curve started to flatten coinciding with the first cracks to form in the specimen, the strains rose sharply and significant tensile stresses developed at the rear side of the shear connectors (Gauge 0), eventually exceeding the concrete tensile strength, i.e. horizontal cracking at the backside of the concrete rib occurred. Note that as concrete has a very low tensile strength, cracking is experienced at around $100\mu$-strain. The remaining increase in strains shown in the graph represents the growth of this crack as the masonite piece to which the gauges were glued was still able to deform after the formation of the crack. Eventually, the masonite fractured causing the failure of the gauge. The gauges spaced 100mm transversely from the centreline of the specimen only start to show a significant increase in measured strains once the centreline gauge experienced strains of a magnitude resembling the concrete tensile strength and, in a similar manner, the gauges spaced 200mm transversely from the centre only indicated an increase in strains once the 100mm
gauges had reached strains indicating that the concrete tensile strength had been exceeded. The readings highlight how the horizontal crack in the concrete rib propagated from the vicinity of the shear connection transversely to the outer edges of the specimen.

**Figure 4-41:** Concrete strain-gauge readings between concrete rib and cover slab for a narrow, conventionally-reinforced stud pair specimen (RIB2-SWP2)

Once the transverse cracks had propagated to both transverse edges of the concrete slab, the load dropped rapidly as there was no further redistribution of the tensile stresses possible. The reason that this particular specimen did not fail completely, and in an even more brittle manner as experienced by other researchers (Oehlers and Lucas 2001; Patrick 2000), was probably due to the use of bottom longitudinal reinforcement bars placed on top of the steel decking. After failure, the bars were still embedded in the front and back half-rib of the test specimen which were otherwise unloaded. A proportion of the longitudinal force could consequently be transferred via the longitudinal bars into these fringe areas. In a beam application where shear connectors are placed in every pan, these fringe areas do not exist. After testing, the two longitudinal bottom reinforcement bars were cut close to the two half-ribs. The solid part then separated easily from the concrete ribs and the shear connectors, revealing a nearly horizontal failure surface which had only locally progressed to the underside of the head of the studs (Figure 4-42 and Figure 4-43).
As the failure surface passed completely above or along the bottom reinforcement mesh, any different bar spacing or positioning would have still rendered it ineffective.

Figure 4-42: Separation of concrete ribs from solid cover part after cutting longitudinal bottom reinforcement

Figure 4-43: Rib shearing failure surface after cover part was removed by hand

Rib shearing failures have previously been observed in shear connections located close to free concrete edges as apparent in composite edge beams (Patrick and Bridge 2000). However, in the tests described in Chapter 4.3.2, rib shearing failures were not only observed in specimens with narrow concrete slabs but also in specimens with up to 1200mm wide slabs where pairs of studs where used, or in shear connections incorporating the stud enhancing devices.

4.4.5 Stud pull-out failure

This failure mode has similar characteristics to the rib shearing failure. It is characterized by the solid cover part of the concrete slab separating from the shear connectors surrounded by cones of concrete (Figure 4-44). This failure mode can also be very brittle in nature when no reinforcement passes through the potential failure surface. In many cases, the cover part of the concrete slab could simply be separated from the remaining specimen once the test was stopped (Figure 4-44). The observed failure surface in the longitudinal direction usually propagated from the top corner of the steel decking, passing to the underside of the stud heads and then to the top corner of the adjacent rib. Usually no transverse reinforcement crossed the failure
surface of the concrete cone as the surface had simply propagated below the bottom reinforcement mesh placed on top of the steel decking.

![Figure 4-44: Separation of cover slab from shear connectors and steel decking after stud pull-out failure occurred (Specimen P02-P2)](image)

For the specimens where the steel decking was omitted, it was observed that two diagonal cracks formed behind the stud connectors at the top rear corner of the concrete rib from where they propagated downwards and, once at its bottom, horizontally along the underside of the concrete rib (Figure 4-45).

The failure mode was generally observed for specimens incorporating sufficiently wide concrete slabs, either when the stud enhancing device was used in both, single stud and stud pair applications, or for conventionally reinforced stud pair connections in KF70® rib geometry. Also, some of the conventionally reinforced shear connections incorporating stud pairs in W-Dek® rib geometries where wider concrete slabs were tested experienced combined rib-shearing and stud pull-out failure modes.

The concrete gauge readings (Figure 4-46) indicate that, similar to rib shearing failure, this failure mode was also triggered by horizontal cracking in the concrete slab at the rear side of the shear connectors from where the crack started to propagate transversely across the concrete rib. In contrast to rib shearing failure, the cracks propagated diagonally down the concrete rib leaving the outer edges of the ribs
relatively stress-free as indicated by the low strain measurements in the outer gauges (200L, 400L).

**Figure 4-45:** Development of a stud pullout crack (Specimen RIB3-SWR1)

**Figure 4-46:** Concrete strain-gauge readings between concrete rib and cover slab for a 1200mm wide, stud pair specimen with steel decking omitted (RIB3-SWR4)
4.5 Analysis of test results

4.5.1 Solid slab specimens

4.5.1.1 Assessment of current prediction method

Numerous studies into the various factors influencing the behaviour of stud shear connections in solid slabs applications have already been undertaken during the past forty years and these have been summarized (Ernst et al. 2005). It was established that at lower concrete strengths, the concrete properties have an influence on the shear connection capacity while at higher concrete strengths, the shear connection capacity seems to be solely influenced by the shear capacity of the stud shank. This relationship is generally accounted for in the current prediction method firstly derived by Ollgaard et al. (1971) and given in Equations (2-1) and (2-2) in Chapter 2.2.2.

However, much of the test data used to derive the shear connector strength in previous assessments were obtained using early push-out test methods which did not comply with the standard test method described in Appendix B of Eurocode 4 (CEN 2004). These could have yielded a different behaviour and led to premature failures of the shear connection (see also Chapter 3.1.1). It was therefore decided to undertake a new assessment of the solid slab shear connector strength based on a total of 81 push-out tests performed over the past 15 years by various researchers. In addition to the solid slab tests given in Ernst et al. (2005), further tests on shear connections in high-strength concrete (An and Cederwall 1996; Döinghaus 2002) and on 16mm and 19mm diameter studs (Gnanasambandam 1995) have also been included in this assessment. The tests included shear connections with concrete strengths between $17\text{MPa} \leq f'_c \leq 97\text{MPa}$, stud diameters of $16\text{mm} \leq d_{bs} \leq 25\text{mm}$ and stud material strengths of $400\text{MPa} \leq f_{uc} \leq 550\text{MPa}$. For many specimens, only the nominal stud material tensile strength rather than a measured one was given. In deriving the results where no concrete elastic modulus was measured, the relationship defined in Eurocode 2 (CEN 1992) was assumed.
From the test results, values of $c_{1m} = 0.44$ and $c_{2m} = 0.93$ were determined for the mean stud strengths in accordance with Equation (2-1) and (2-2). The comparison of the measured stud strengths with their predicted capacity is shown in Figure 4-47. The relationship given seems to provide a satisfactory prediction of the shear connector strength, even for shear connections including large stud diameters or high-strength concrete. However, it should be kept in mind that high-strength concrete has a reduced deformation capacity and the shear connections usually do not fulfil the ductility criterion of Eurocode 4 (CEN 2004) such that they do not provide a characteristic slip capacity of 6mm. The work of Döinghaus (2002) investigated the influence of high-strength concrete on the shear connection ductility in solid slab applications in depth.

![Predicted vs Experimental stud capacity comparison](image.png)

**Figure 4-47**: Comparison of the shear connector strength of 81 solid push-out tests with their predicted mean capacity

In order to illustrate the main influence factors in shear connection in solid slabs, the tests results of the above-mentioned push-out tests and their theoretical predictions are plotted as a function of the concrete strength in Figure 4-48. The concrete compressive strength only has an influence on the shear connection strength up to a certain limit above which it is influenced by the stud material strength. This limit is typically between 30MPa to 40MPa for commonly used shear connector materials. An increase in stud material strength, on the other hand, only increases the strength...
of the shear connection above this particular limit. However, a change in the
diameter of the stud shank generally influences the shear connection capacity for all
concrete strengths. Note that for stud diameters of 22mm and more it appears that the
equations slightly underestimate the shear connector capacity while the results
generally agree well with studs of 19mm diameter or less.

![Graph showing the influence of concrete strength on shear connector capacity in solid slabs.](image)

**Figure 4-48:** Influence of concrete strength on shear connector capacity in solid slabs

### 4.5.1.2 Stud configuration

No significant difference in strength can be observed in the CTR-RIB series when
the test results of the solid slab single stud specimens are compared to the stud pair
specimens (Figure 4-49). In all specimens, the condition of the minimum transverse
stud spacing in accordance with Eurocode 4 (CEN 2004) was fulfilled. The stud pair
specimens seem to have a slightly lower ductility as measured by the stud load at
6mm slip, but the total deformation capacities were similar to the single stud
specimens. Both experienced sufficient slip of around 11-12mm without any
reduction in load capacity. Ultimately, similar types of stud shearing failures were
observed accompanied by sharp drops in load.
4.5.1.3 Friction at steel-concrete interface

Some controversy has recently arisen about the influence that friction, which may develop at the interface between the concrete slab and steel beam, has on the strength of the shear connection. In Lungershausen (1988), it was postulated that in order to balance the tensile forces which develop in the shank of the stud, compressive stresses develop under the head of the stud in the concrete slab which result in normal compressive forces at the steel-concrete interface that activate additional frictional forces (see also Chapter 2.2.1). A similar argument was also proposed by Rambo-Roddenberry et al. (2000). There, a push-out test series was presented where friction between steel beam and concrete slab was varied by placing a flat galvanized steel sheet at the interface and, in another test, by additionally applying grease to the interface. The results (as shown in Table 3-1) showed an increase in strength of nearly 20% from a greased steel sheeting interface to an untreated steel-to-concrete interface. However, these tests were conducted while a normal compressive force in the order of 10% of the total shear force was applied to the concrete slab which ensured the activation of friction. In contrast, for the specimen consisting of an untreated steel-concrete interface to which no normal forces were applied, a similar strength was reached as for the specimen with the minimized frictional coefficient (greased sheeting-concrete interface) and applied normal forces. This would indicate
that no additional friction is activated in the standard push-out test. In tests performed by Scheele (1991), an influence of interface friction was also dismissed as in some specimens, the concrete slabs separated from the steel section along their complete interface while the shear connection strengths were of similar magnitudes as in specimens where this separation did not occur. X-ray photographs of tests were additionally presented where the complete separation of steel flange and concrete slab at the latter stages of the tests can be clearly seen.

The test results shown in Figure 4-49 are very much in agreement with Scheele (1991) as the specimen which had a Teflon layer provided between the steel-concrete interface, hence very little frictional resistance, showed virtually no difference in strength and behaviour compared to the specimen with the untreated interface. During the course of testing, it was also observed that the concrete slab completely separated from the steel section when the maximum capacity was approached (Figure 4-50). Note that the front part of the concrete slab experienced a larger amount of uplift than the rear part which caused rotation of the slab. This larger uplift in the front of the specimen might have been partly introduced by the flexible vertical restraint of the test set-up (see Chapter 3.3). However, the limited vertical stiffness of the shear connectors experiencing tensile forces is thought to have also contributed to the uplift of the entire slab.

![Uplift of concrete slab from steel flange during testing (RIB4-SWD9)](image_url)
4.5.1.4  **Tensile forces in shear connectors**

The axial forces in the shanks of all studs were measured in the test specimens RIB4-SWD9 and RIB4-SWD10 and are compared with the horizontal load slip behaviour of the particular shear connections in Figure 4-51. The shank of each stud was generally subject to tensile force which increased proportionally with the applied shear force. However, the tensile forces remained comparatively low, at its peak reaching 7% of the applied shear force in specimen SWD9 and 12% in specimen SWD10. Eurocode 4 (CEN 2004) requires the shear connector to resist 10% of its shear force capacity in tension which seems to reflect the force distribution observed in the solid slab test specimens well. Conversely, it can also be concluded that the boundary conditions of the horizontal test set-up are suitable for simulating composite beam behaviour even though vertical movement of the concrete slab is not completely resisted.

![Graph showing average tensile forces in shank of stud connectors compared with horizontal shear forces](image)

**Figure 4-51:** Average tensile forces in shank of stud connectors compared with horizontal shear forces

According to Scheele (1991), tensile forces in the shank of a stud connector only start to have a weakening effect on the shear capacity once they exceed 10% of its tensile capacity provided the embedment depth into the surrounding concrete is
sufficient so that stud pull-out failure does not occur. The measured maximum axial force in the test specimens ranged between 8% and 12% of the tensile shank capacity, hence its effect is considered minimal.

Further observations to be made from the measurement were that the rear studs initially seemed to experience a larger amount of tensile axial force, but during the further course of testing, these axial forces distributed more evenly between the front and rear stud rows. It also appeared that the shear connection which experienced the largest amount of tensile force failed first, whether it was the front or rear row of shear connectors. The specimen with the Teflon layer seemed to experience slightly higher axial tensile forces in the shank of the studs.

4.5.1.5 Summary

The most common method for determining the stud strength in solid slab applications, derived by Ollgaard et al. (1971) which is also used for the shear connection design in AS 2327.1 (Standards Australia 2003a), was found to include the major factors influencing the solid slab shear connection capacity which are the concrete strength, the area of the stud shank and the stud material strength. The method generally provides a good prediction of the expected shear connection capacity even if high strength concrete or larger stud diameters are used. Within its application limits, the stud configuration does not seem to have an impact on the shear connection strength and behaviour as long as sufficient transverse bottom reinforcement is provided.

For the numerous push-out test results published by various researchers where, in some cases, the test methods differed significantly from the standard push-out tests given in Eurocode 4 (CEN 2004), the evaluation showed that the results generally fitted into the same schema. Consequently, stud shear connections in solid slab applications seem to be a fairly robust system where the test method only has a limited effect on the connector strength provided the mode of failure is not altered or additional frictional forces are activated. As a consequence, a high level of repeatability can be expected.
When the new horizontal push-out test rig described in Chapter 3.3 was used, tensile forces generally developed in the shank of the shear connectors which seemed to contribute to the separation of the concrete-steel interface of the test specimens when the maximum capacity was approached. The self-weight of the concrete slab was also found not to suppress the effect of separation in the horizontal push-out tests. Hence, friction does not seem to contribute to the stud capacity in the new test rig. As the stud capacities determined from these tests match well with test results obtained from double-sided push-out tests performed by other researchers, it can also be concluded that the influence that friction has in these double-sided tests is also insignificant. Nevertheless, a more substantial frictional component may appear where horizontal restraints or lateral compressive forces are additionally applied (see also Chapter 3.1.1.5).

4.5.2 Specimens incorporating profiled steel decking

4.5.2.1 General

In comparison to solid slab applications, the number of failure modes and material and geometric parameters in secondary beam applications is much larger. This makes it nearly impossible to investigate all the various factors and their interactions thoroughly, in particular when the large number of ever-changing steel decking geometries is taken into consideration. To aggravate the situation, there is no agreement between the various researchers about a common push-out test method which itself might influence the behaviour of the shear connection significantly (see Chapter 3.1.2). Comparisons with test results obtained by different researchers can therefore only be made with great care. Consequently, this analysis focuses mainly on the particular rib geometries tested which are characterized by relatively wide-open trapezoidal concrete ribs and steel decking heights between 50mm-80mm (2in.-3in.). Test results of other researchers were only included if similar types of steel decking geometries were used.
In the following, the influence of the various parameters and elements, which were investigated during the course of the numerous test series on the behaviour of the shear connection in a secondary composite beam application, are summarized and discussed.

### 4.5.2.2 Concrete tensile strength

Concrete cracking is the initiator of each of the premature failure modes. The concrete tensile strength is therefore thought to be one of the main factors influencing the shear connection strength. In accordance with the latest draft of the revision of AS3600-2001 (Standards Australia 2005), the mean concrete tensile strength \( f_t \) can be determined in relation to its cylinder compressive strength \( f'_c \) from

\[
    f_t = 0.5 \sqrt{f'_c}.
\]  

(4-1)

Figure 4-52 and Figure 4-53 show the influence that the concrete tensile strength has on the maximum shear connector strength and on the strength at 6mm slip in the two rib geometries investigated. It becomes evident that the capacity of the shear connection increases with increasing concrete tensile strength at both maximum load and at 6mm slip. The behaviour of the specimens can be reasonably well predicted with a linear trendline. Comparing the specimens in Figure 4-52, the increase in strength seems to be similar for conventional reinforced specimens, specimens including the ring enhancing device and specimens having cross-wires in the concrete rib. For the stud pair specimens which also included the waveform reinforcement element, the increase in maximum strength is much smaller and at 6mm slip there is hardly any change in strength. Hence, the waveform element seems to make the connection less dependent on the concrete tensile strength as it controls the cracking after initiation. For the specimens where cross-wires were provided in the concrete rib, the trendline of the strengths at 6mm slip was not drawn as the specimen which showed an unusual low strength (DM01-S07) had insufficient welds and might have failed prematurely.
Figure 4-52: Influence of concrete tensile strength on shear connection strength in KF70\textsuperscript{®} rib geometry at maximum load $P_{\text{max}}$ and at 6mm slip $P_{6\text{mm}}$.

Figure 4-53: Influence of concrete tensile strength on shear connection strength in W-Dek\textsuperscript{®} rib geometry at maximum load $P_{\text{max}}$ and at 6mm slip $P_{6\text{mm}}$.

4.5.2.3 Stud position in concrete rib

Steel decking geometries with a central stiffener located in the centreline of the pan have become more common over recent years. This arrangement prevents the studs
from being placed in the central position of the concrete rib. Numerous researchers have already investigated the effect that the stud position in a pan has and it is generally acknowledged that the positioning of the shear connector in a position where the concrete bearing zone in front of the shear connection becomes larger is the preferred option. The shear connection strengths of push-out tests performed by different researchers where the stud position was varied (Mottram and Johnson 1990; Rambo-Roddenberry 2002; van der Sanden 1996; Yuan 1996) are compared to their particular position in Figure 4-54. The influence that the longitudinal distance of the shear connector to the edge of the concrete rib in the direction of the longitudinal shear, $e$, has on the maximum stud strength can be clearly seen. A similar influence of the stud position on the load at 6mm slip can also be observed for most of the specimens noting that the maximum stud strength is generally reached at slips of about 2mm. However, for some specimens, the reduction in strength seems to be very severe, even if comparatively large edge distances were provided. One specimen actually failed before a deformation of 6mm slip could be reached.

**Figure 4-54:** Influence of edge distance on maximum load $P_{\text{max}}$ and load at 6mm slip $P_{\text{6mm}}$ – tests by various researchers

Most of the single stud specimens in secondary beam applications experienced rib punch-through failures for which it can be assumed that an increase in the shear connector distance from the free longitudinal concrete edge results in an increase in
the size of the concrete wedge which breaks out of the concrete rib at maximum load (see Chapter 4.4.2). A larger concrete wedge will provide a higher capacity against complete failure as its failure surface area is increased. For stud connectors used in fastener applications located close to a free longitudinal concrete edge and subject to shear forces, the formation of similar types of concrete wedges has been observed. It was found that the edge distance $e$ increased the fastener strength by a factor of about $e^{1.5}$ (Eligehausen and Mallee 2000).

![Specimen P03-SR3 after failure](image)

**Figure 4-55:** Specimen P03-SR3 after failure (cut longitudinally and steel decking partially removed)

The geometry of the KF70® steel decking geometry is such that every other pan has a stiffener along its centreline. In the tests described in Chapter 4.2.1, the stud placements were in central position in ribs without a lap joint and 30mm from the centreline in the designated longitudinal direction in the ribs consisting of a lap joint (see also Figure 4-1). In some of these tests, it was observed that a concrete wedge which characterizes the rib punch-through failure had formed in front of the stud that was placed in the central position of the pan while the other stud, which was placed in the favourable position, showed no signs of such concrete cracking and the connector simply had sheared off above its weld (Figure 4-55). Figure 4-56 shows the graph of this particular test (P03-SR3) where no sudden drop in load, which is typical for a rib punch-through failure, could be observed. It is assumed that after the rib punch-through failure of the centrally located stud occurred, its loss in strength was compensated by a redistribution of the shear forces to the favourably located stud which eventually experienced a stud shearing failure under the increased shear forces. However, another test specimen (DM01-S12) which had the same features
and experienced a similar shear connection strength behaved in a far more brittle fashion as it experienced rib punch-through failures in both shear connections (Figure 4-56). Consequently, a ductile behaviour of conventionally reinforced shear connections incorporating the KF70® decking geometry should not be relied on where every second stud is placed in a favourable position.

![Load-slip behaviour of two similar single stud specimens incorporating KF70® steel decking geometry](image)

**Figure 4-56:** Load-slip behaviour of two similar single stud specimens incorporating KF70® steel decking geometry

For stud pairs, investigations into stud placement in a concrete pan have rarely been reported in the literature. Stud pairs are usually required to be placed in a staggered position if a stiffener in the centre of the rib is apparent or in the central position otherwise, e.g. Eurocode 4 (CEN 2004). As part of the KF70® geometry push-out tests reported in Chapter 4.2.1, the positions of the studs in the pan with a lap joint were varied between a diagonal staggered position and a favourable side position but only in specimens where a waveform reinforcement element was provided. There, no significant influence of the stud configuration on the shear connection behaviour could be observed (Figure 4-57). In the push-out tests incorporating the W-Dek® geometry the stud position was varied between a central and a diagonal (staggered position) in a conventional reinforced stud pair application. Both specimens experienced similar shear connection strengths while the specimen with the stud pairs placed in a central position experienced an increased deformation capacity (Figure 4-57). However, the central positioned specimen additionally had the steel
sheeting anchored at the transverse edges of the concrete slab (see Figure 4-7) which might have also accounted for the improved ductility. Further investigations into the influence that the stud positioning has on the behaviour of stud pair shear connections would certainly be desirable.

![Graph showing influence of placement of stud pairs in concrete ribs](image)

**Figure 4-57:** Influence of placement of stud pairs in concrete ribs

### 4.5.2.4 Number and transverse spacing of shear connectors

In secondary beam applications, unlike in solid slab applications, the increase in the number of shear connectors per concrete rib does not result in an equal increase of the shear connection strength. Generally, it can be said that the number of studs per rib increases the overall load in the individual concrete rib and makes it more prone to the brittle concrete-related failure modes.

In the case of rib punch-through failures, the concrete wedges, which would theoretically form individually in front of each shear connector, overlap as the two shear connectors are spaced closely together (Figure 4-58). As a result, the failure wedge does not have double the fracture area and hence fails at a comparatively lower load compared to that for two single independent studs.
The shear connection behaviour might even be altered altogether if a second stud is introduced. Figure 4-59 compares the behaviour of two 600mm wide specimens with a W-Dek concrete rib geometry where the steel decking was omitted; one specimen had a single stud arrangement, the other one a stud pair arrangement. While the single stud specimen experienced ‘typical’ rib punch-through behaviour, the specimen involving stud pairs ultimately failed in rib shearing. The load-slip behaviours differed considerably for these specimens and the load per stud connector was considerably less in the stud pair specimen. Also, the stud pair specimen did not experience a second rise in strength associated with a bending of the stud. Figure 4-59 also shows the concrete gauge measurements at the rear side of the concrete rib at the centreline of the specimen which highlights the different stress states. While initially both specimens experienced similar strains, the further increase in load in the concrete rib of the stud pair specimen resulted in the development of very high vertical strains which eventually resulted in the development of the horizontal rib shearing cracks. These types of cracks did not occur in the single stud specimen as the vertical strains remained constant after the rib punch-through failure had developed.
**Figure 4-59:** Influence of number of studs on load slip behaviour and concrete gauge reading for W-Dek® geometry specimens with narrow concrete slabs

**Table 4-23:** Effects of a second shear connector in a concrete rib on strength and ductility in specimen incorporating W-Dek® geometry

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Increase max. strength / concrete rib $\Delta P_{\text{max}, \text{rib}}$ [kN]</th>
<th>Ductility of specimen $P_{\text{max}} / P_{\text{max}}$</th>
<th>Failure mode$^1$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>[kN] [ %]</td>
<td>Single studs [ %] Stud pairs [ %]</td>
<td>Single stud</td>
</tr>
<tr>
<td>No steel decking, conventionally reinforced, wide slab</td>
<td>47 [58]</td>
<td>61-65 62-77</td>
<td>RPT RPT</td>
</tr>
<tr>
<td>No steel decking, conventionally reinforced, narrow slab</td>
<td>25 [33]</td>
<td>65-75 63-73</td>
<td>RPT RPT/RS</td>
</tr>
<tr>
<td>Steel decking, conventionally reinforced, wide slab</td>
<td>51 [51]</td>
<td>94 89-95</td>
<td>RPT RPT</td>
</tr>
<tr>
<td>No steel decking, spiral enhancing device, wide slab</td>
<td>48 [44]</td>
<td>87 73</td>
<td>RPT/SP RPT/SP</td>
</tr>
<tr>
<td>No steel decking, spiral enhancing device, narrow slab</td>
<td>47 [57]</td>
<td>66 55</td>
<td>RPT/RS RPT/RS</td>
</tr>
<tr>
<td>Steel decking, spiral enhancing device, wide slab</td>
<td>33 [29]</td>
<td>85 94</td>
<td>RPT/SS RPT/SP</td>
</tr>
<tr>
<td>Steel decking, spiral enhancing device, narrow slab</td>
<td>28 [28]</td>
<td>80 78</td>
<td>RPT/RS RPT/RS</td>
</tr>
<tr>
<td>Steel decking, spiral enhancing device, waveform element, wide slab</td>
<td>53 [38]</td>
<td>93 98</td>
<td>RPT/SS RPT</td>
</tr>
<tr>
<td>Steel decking, spiral enhancing device, waveform element, narrow slab</td>
<td>51 [42]</td>
<td>94 93</td>
<td>RPT/SS RPT</td>
</tr>
</tbody>
</table>

$^1$: RPT: Rib punch-through, SS: Stud shearing, RS: Rib shearing, SP: Stud pull-out

In Table 4-23, the effects that the application of a second shear connector in a concrete rib had on the strength and ductility are summarized for the tests
incorporating the W-Dek® geometry. In some specimens, the increase in strength due to a second stud was as low as 30% which makes the application of stud pairs very uneconomical as an additional 100% of material is added. The average increase in strength of all tests was 41% which agrees well with the provision given in Equation (2-8) where an increase of $\frac{2}{\sqrt{n_s}} = 41\%$ is assumed. However, if the results are more differentiated, it can be seen that for the specimens which experienced a change in their failure mode, the increase in strength was generally lower as the different failure mode weakened the specimen. If these specimens are excluded, the total amount of increase seems to remain nearly constant, between 47kN-53kN, for all but one specimen which experienced a brittle rib shearing failure in both, single stud and stud pair applications. Hence, the increase in strength for the application of a second stud in a shear connection appears to be independent of the existence of steel decking, the width of the specimen (if wide enough that the wedges can develop) or the inclusion of stud enhancement devices and waveform reinforcement elements if no change in failure mode occurs. However, if the second shear connector triggers additional failure modes such as stud pull-out or rib shearing, not only is the increase in strength reduced, but also the ductility can be altered (see also Figure 4-59).

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Increase maximum strength / concrete rib $\Delta P_{\text{max,rib}}$</th>
<th>Ductility of specimen $P_{\text{max}} / P_{\text{max,rib}}$</th>
<th>Change of failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$[\text{kN}]$</td>
<td>$[%]$</td>
<td>Single studs $[%]$</td>
</tr>
<tr>
<td>UWS tests: Steel decking, $t=1.0\text{mm}$, cross-wire in concrete rib, studs in dia/ctr position, $s_x=80\text{mm}, f'_c=32\text{MPa}$</td>
<td>15</td>
<td>11</td>
<td>98</td>
</tr>
<tr>
<td>Patrick tests: No steel decking, studs in central position, $s_x=80\text{mm}, f'_c=33\text{MPa}$</td>
<td>35</td>
<td>47</td>
<td>71</td>
</tr>
<tr>
<td>Patrick tests: No steel decking, ring device, studs in central position, $s_x=80\text{mm}, f'_c=33\text{MPa}$</td>
<td>62</td>
<td>69</td>
<td>91</td>
</tr>
<tr>
<td>Oehler/Lucas tests: Steel decking, $t=0.75\text{mm}$, studs in unfav/ctr position, $s_x=57\text{mm}, f'_c=47\text{MPa}$</td>
<td>28</td>
<td>34</td>
<td>88-100</td>
</tr>
</tbody>
</table>

**Table 4-24**: Effects of a second shear connector in a concrete rib on strength and ductility in specimen incorporating KF70® geometry
Far less results have been obtained for the application of the KF70® steel decking geometry. Apart from one set of tests performed at UWS (see also Chapter 4.2.1), the results of push-out tests performed by Patrick (2002c) as presented in Chapter 2.2.4 and by Oehlers and Lucas (2001) are considered in Table 4-24. It should be noted that the tests of Patrick (2002c) might overestimate the increase in strength as it only consisted of a half-concrete rib which might have provided a higher resistance against some brittle failure modes such as rib shearing or stud pull-out. There is no conclusive trend as a change of failure mode was observed in all the tests when a second shear connector was added. However, the increase in strength was found to be as low as only 10% for a stud pair connection compared to a single stud connection and the ductility of the specimens was, in some cases, significantly reduced.

**Figure 4-60:** Behaviour of stud pair specimens including the ring enhancement devices with different transverse spacings $s_x$

The transverse spacing $s_x$ of the shear connector was only altered in one of the tests described in Chapter 4.2 where it was increased from 80mm to 120mm. Compared to the specimens of an otherwise similar lay-out, no increase in strength was observed as shown in Figure 4-60. However, these tests did experience stud pull-out failures. For shear connections where rib punch-through failures occur, there might be an increase in strength as the failure surface of the concrete wedge would become larger.
(see Figure 4-58). Such an increase is typically experienced in fastener applications for studs positioned close to a free concrete edge in longitudinal direction (Eligehausen and Mallee 2000).

4.5.2.5 Steel decking

As part of the systematic investigation into the shear connection behaviour incorporating the W-Dek® decking geometry, numerous specimens had the steel decking completely omitted. The load slip behaviours of some of these specimens are shown in Figure 4-61. It can be seen that the existence of steel decking did not generally alter the behaviour of the specimen, but it did increase the strength and ductility of the conventionally reinforced specimens which experienced rib punch-through behaviours. The initial stiffness for these specimens did not differ significantly from the specimens including the decking before the load-slip curve started to flatten which is a strong indication that the pre-holed steel decking only started to have a significant effect once the cracks leading to rib punch-through failure started to occur. The beneficial effects of the steel decking seemed to be stronger where a thicker steel sheeting was used. In contrast, no increase in strength or ductility was observed for the specimens with the spiral enhancing device which reached a comparatively higher load per concrete rib but experienced more brittle stud pull-out behaviours. This is an indication that beneficial effects of the steel decking might only be restricted to certain failure modes.

The results for all specimens where the influence of the steel decking was investigated are summarized in Table 4-25. The steel decking generally had a beneficial effect for shear connections which experienced pure rib punch-through failures whereas for combined rib punch-through and rib shearing failures, the beneficial effects were not always observed, in particular where large loads per concrete rib existed. However, the transverse anchorage of the steel decking (see Figure 4-7), which was also applied to the specimens with narrow concrete slabs, might not have reflected the case of actual edge beam applications and might have overstated the beneficial effects of the steel decking. Where stud pull-out failures were experienced, the existence of the steel decking did not have a significant effect
on the specimen strength. For all specimens, the increase in strength generally became larger at higher slips as indicated by the values of the increased strength at 6mm slip.

![Graph showing load-concrete rib vs slip for different specimens.](image)

**Figure 4-61:** Influence of W-Dek® steel decking in specimens with wide concrete slabs

**Table 4-25:** Effect of existence of steel decking in W-Dek® specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Increase maximum strength / concrete rib $\Delta P_{rib,\ max}$ [kN]</th>
<th>%</th>
<th>Increase strength at 6mm slip / concrete rib $\Delta P_{rib,\ 6mm}$ [kN]</th>
<th>%</th>
<th>Failure mode$^1$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single stud, conventionally reinforced, wide slab$^2$</td>
<td>16</td>
<td>19</td>
<td>38</td>
<td>65</td>
<td>RPT</td>
</tr>
<tr>
<td>Stud pairs, conventionally reinforced, wide slab$^2$</td>
<td>22</td>
<td>18</td>
<td>42</td>
<td>43</td>
<td>RPT</td>
</tr>
<tr>
<td>Stud pairs, conventionally reinforced, narrow slab$^2$</td>
<td>36</td>
<td>35</td>
<td>45</td>
<td>70</td>
<td>RPT/RS</td>
</tr>
<tr>
<td>Single stud, spiral enhancing device, wide slab$^2$</td>
<td>7</td>
<td>6</td>
<td>14</td>
<td>15</td>
<td>RPT/SP</td>
</tr>
<tr>
<td>Single stud, spiral enhancing device, narrow slab$^2$</td>
<td>19</td>
<td>23</td>
<td>27</td>
<td>49</td>
<td>RPT/RS</td>
</tr>
<tr>
<td>Stud pairs, spiral enhancing device, wide slab$^2$</td>
<td>-8</td>
<td>-5</td>
<td>10</td>
<td>9</td>
<td>RPT/SP</td>
</tr>
<tr>
<td>Stud pairs, spiral enhancing device, narrow slab$^2$</td>
<td>0</td>
<td>0</td>
<td>15</td>
<td>41</td>
<td>RPT/RS</td>
</tr>
</tbody>
</table>

$^1$: RPT: Rib punch-through, SS: Stud shearing, RS: Rib shearing, SP: Stud pull-out  
$^2$: sheeting thickness $t=0.75\text{mm}$,  
$^3$: sheeting thickness $t=1.00\text{mm}$
For single stud applications incorporating the KF70® decking geometry and where the studs were directly welded through the steel decking, an increase in shear connection strength was also observed with an increase in steel decking thickness, as obtained from the tests described in Chapter 4.2.1 (Figure 4-62). Push-out tests by van der Sanden (1996), where the studs were also welded through the steel decking (except for the one case where the decking was completely omitted), indicated similar influences of the steel decking thickness on the shear connection strengths (Figure 4-62).

![Figure 4-62: Influence of sheeting thickness on maximum shear connection capacity where the studs were welded through the decking](image)

**4.5.2.6 Concrete slab width**

The influence of the concrete slab width on the shear connection strength and on the failure mode experienced in the push-out tests described in Chapter 4.2.2 is shown in Figure 4-63 for single stud specimens and in Figure 4-64 for stud pair specimens. For conventionally reinforced single stud applications which experienced rib punch-through behaviour, the width was found to have only a minimal impact on the shear connection strength. If, on the other hand, the spiral enhancing device was used, the failure mode changed from a rib-shearing failure in narrow specimens to a stud pull-out failure for the wider specimens which also resulted in an increase in the shear...
connection strength. In the two specimens where rib-shearing and stud pull-out failures were controlled through the use of the waveform reinforcement element combined with the spiral enhancing device, similar failure modes were experienced for both widths, but the strength was still lower for the narrow concrete slab application.

**Figure 4-63:** Influence of concrete slab width on shear connection strength and failure mode for single stud specimens incorporating W-Dek® geometry

**Figure 4-64:** Influence of concrete slab width on shear connection strength and failure mode for stud pair specimens incorporating W-Dek® geometry
In stud pair applications, rib-shearing failures were generally experienced for specimens with a narrow concrete slab unless the failure mode was suppressed by providing the waveform reinforcement elements. However, even specimens with concrete slab widths of up to 1200mm experienced rib-shearing failures in some concrete ribs which generally resulted in reductions in shear connection strengths. If the specimen width was further increased, the failure modes changed to stud pull-out or rib punch-through depending on the shear connection layout. Again, the shear connection strengths of otherwise similar specimens were found to increase slightly with increasing specimen widths.

In Figure 4-65, the vertical concrete gauge readings between the concrete rib and solid cover slab at the rear side of the shear connections are compared for different concrete slab widths in conventionally reinforced stud pair applications. It can be seen that for a given slip, the vertical concrete strains are higher for the narrower slabs. Specimens RIB2-SWP1 to SWP3 all experienced rib-shearing failures. In the narrowest, the 400mm wide specimen (RIB2-SWP3), the horizontal rib shearing crack seemed to have developed at the lowest slips. In the 900mm wide specimen (RIB2-SWP1), one side of a concrete rib actually experienced a diagonal stud pull-out crack while the other side and both sides in the second concrete rib displayed rib shearing cracks. Despite the different strain levels in these three specimens, the ultimate failure load remained essentially constant (see Figure 4-23). A further increase in width to 1200mm suppressed the rib-shearing failure and the specimen (RIB3-SWR1) experienced the development of a diagonal concrete crack in one rib which characterizes a stud pull-out failure. The diagonal crack eventually stopped propagating when the load dropped due to rib punch-through failure. Hence, the strain level on the masonite piece, which represents the concrete crack width, remained constant. No diagonal stud pull-out cracks were observed on the 1800mm wide specimen (RIB3-SWR2) and the measured strains at the rear side of the concrete rib remained below the value of strain corresponding to the concrete tensile strength.
In consequence, the influence of the concrete slab width on the shear connection strength and behaviour is more severe than initially assumed, in particular where stud pairs are tested. If an internal beam application is to be simulated, the concrete slab of the test specimen should be of sufficient width to ensure transverse distribution of the vertical stresses which develop at the rear side of the concrete ribs and, hence, to avoid potential rib shearing failures which were found to result in reduced shear connection strengths and ductility. In the case of the W-Dek® geometry, it was found that a concrete slab of 1800mm width was required to model an internal beam application if stud pairs are tested. On the other hand, the results of push-out tests obtained from specimens with wider concrete slabs should not be applied where shear connections close to a free concrete edge are to be considered as this application would experience a different failure mode with much reduced strength and ductility.

In the tests on the KF70® decking geometry, as described in Chapter 4.2.1, the concrete slabs were generally only 600mm wide and additional edge waveform reinforcement was provided in order to suppress the effects of rib shearing failures. Consequently, some of the test results might provide a lower, conservative estimate
for KF70® internal composite beam applications with wider concrete slabs, in particular for the stud pair connections where predominantly stud pull-out failures were experienced. For the single stud tests which typically experienced rib punch-through failures (unless the ring enhancing device was provided), the relatively small width of the concrete slab can be assumed not to have had a significant impact on the shear connection behaviour.

Shear connections close to a free concrete edge typically occur in composite edge beams where only one side of the connection is close to the edge while the other side adjoins a wide internal slab. As part of the investigations described in Chapter 4.2.2, specimens of an eccentric layout were tested where the concrete slab was such that it had an outstand from the shear connection centreline of 300mm on one side and 900mm on the other side. The results of these tests are compared in Figure 4-66 to similar symmetrical 600mm wide specimens which provided a slab outstand of 300mm on both sides of the shear connection centreline.

**Figure 4-66:** Comparison of push-out test results of stud pair shear connections with an eccentric slab lay-out to similar connections with a symmetrical slab lay-out

For the specimens RIB4-SWD7 and RIB5-SDM4 where rib-shearing failures were suppressed with the help of the waveform reinforcement element, an almost identical behaviour was observed for the eccentric specimen as in the symmetrical narrow concrete slab specimen. Failure in both cases was by rib punch-though. The
conventionally reinforced eccentric specimen (RIB5-SDM3) experienced a similar strength but an improved deformation capacity compared to the conventionally reinforced symmetric specimen (RIB4-SWR6). The reason for the improved ductility in the eccentric specimen is not quite clear as it experienced rib-shearing failure at the connection side closest to the free concrete edge and stud pull-out failure on the other, internal side. From other tests, both these failures are known to usually provide very limited ductility. Therefore this isolated test result should probably not be overrated; particularly as specimen RIB5-SDM3 experienced brittle rib shearing failure in both ribs.

### 4.5.2.7 Bottom reinforcement

In single stud specimens where a bottom layer of welded mesh reinforcement placed on top of the steel decking was provided and where the failure mode was rib punch-trough, no visible effect on the shear connection behaviour was experienced (Figure 4-67). The bottom mesh reinforcement was also not able to prevent rib shearing failure as it generally ran parallel to the potential failure surface which either propagated above or below the reinforcement bars depending on the position of the mesh in the cover slab (see Figure 4-43). The bar spacings of the reinforcement mesh are therefore irrelevant. For concrete pullout failures where failure surfaces between the head of the shear connector and the base of the sheeting pan form, an angle with the horizontal bottom reinforcement is created. In the push-out tests described in this chapter, the failure surfaces were generally found to undermine the bottom layer of reinforcement mesh (see Figure 4-44). However, if the bar spacings of the reinforcement mesh would have been reduced or higher studs would have been used, it might have improved the shear connection behaviour as suggested by Johnson (2005). On the other hand, tests on fasteners loaded in tension, which experienced similar pull-out type of failures, showed that horizontal mesh reinforcement crossing the failure surface did not increase the pull-out strength of the fasteners but was able to improve their ductility if the mesh spacing was small enough to brace the failed concrete cone (Eligehausen and Mallee 2000).
The better option seems to be the application of transverse cross-wires placed in the concrete rib which, in single stud shear connections, were found to increase the shear connection strength about 20%-40% and to reduce the variability in the test results which was experienced otherwise (see Figure 4-19). It is thought that the transverse rib reinforcement is able to control rib splitting failures and to increase the resistance of the triaxial compression zone in front of the shear connector. The transverse reinforcement furthermore reinforces parts of the concrete wedge failure surface in rib punch-through failures and consequently might ensure additional load transfer after break-out of the wedge. In Eligehausen and Mallee (2000), it was shown that the strength of fastener applications which experiences longitudinal splitting failures are dependant on the width of the splitting cracks. Tests on fasteners, loaded in shear and placed in concrete cracks of widths >0.4mm, experienced strength reductions of about 25% compared to fasteners in uncracked concrete. By providing sufficient reinforcement to control the width of the concrete crack, the strength reduction was not as severe and the shear strength could be increased by about 15%-20%. In Oehlers and Park (1992) where shear connections in longitudinally cracked concrete slabs were tested, it was found that the strength of a shear connection in a longitudinally split concrete slab was as low as 60% of its unsplit strength. It was further found that if sufficient transverse reinforcement was provided, a strength of up to 90% of the uncracked shear connection strength could be reached which
reflects an increase of 50%. The results of the push-out tests shown in Figure 4-19 seem to agree reasonably well with these observations although further investigations into the behaviour of transverse rib reinforcement would be desirable.

4.5.2.8 Stud performance-enhancing device

Both stud performance-enhancing devices investigated, viz. the ring device and the spiral device, affected the shear connection behaviour in a similar fashion. The resistance of the shear connection against rib punch-through failure was typically increased. Figure 4-68 shows the rib punch-through failure surface which had a considerably increased width in comparison to a conventional reinforced specimen as it propagated from the outer edges of the ring rather than from the stud. It was also observed that the concrete confined by the enhancing device was virtually uncracked and restricted the movement of the stud base. Hence the base region of the stud was stiffened which reduced the bending effects in the shank of the shear connector and utilized a larger region for the load transfer (see Figure 4-69 to Figure 4-72).

Figure 4-68: Rib punch-through failure surface in single stud shear connection incorporating a ring enhancing device (sheeting cut after failure)
Table 4-26 summarizes the effects that the stud enhancing devices had on the shear connection strength and behaviour. It can be seen that the application of the enhancing device generally increased the maximum strength of shear connections, which initially experienced rib punch-through failures, between 10%-30%. However, the failure mode was also altered to a stud pull-out failure where wider concrete slabs were used or to a rib shearing failure in narrow concrete slabs. Figure 4-73 illustrates the changes in behaviour if the enhancing device was used and the corresponding concrete strain measurements at the rear of the concrete rib. As the spiral enhancing device increased the load applied to the concrete rib, the vertical tensile stresses, which developed between concrete rib and cover slab and which in the initial stages
of loading were of a similar magnitude, were also increased resulting in horizontal cracking of the concrete rib. Hence, by suppressing one failure mode and increasing the shear connection strength, other brittle failure modes were induced. Possible measures to suppress these subsequent failures might be an increase of the stud height or the application of the waveform reinforcing element (as described in Chapter 4.5.2.9). However, higher studs usually require thicker concrete slabs in order to fulfil the concrete cover requirements which could make this measure uneconomical. Even then, in the test series OBD-HS01 where higher studs with no cover were provided, brittle failure modes could still not be completely suppressed. For shear connections which already experienced stud pull-out or rib shearing behaviours (usually stud pair connections), the application of the stud enhancing device did not seem to influence the connection behaviour significantly.

Table 4-26: Effects of application of stud performance-enhancing device

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Increase maximum strength / concrete rib $\Delta P_{\text{rib, max}}$ [kN]</th>
<th>Increase strength at 6mm slip / concrete rib $\Delta P_{\text{rib, 6mm}}$ [kN]</th>
<th>Failure mode$^4$</th>
</tr>
</thead>
<tbody>
<tr>
<td>W-Dek® geometry, no decking, single studs, wide slab</td>
<td>24</td>
<td>43</td>
<td>28</td>
</tr>
<tr>
<td>W-Dek® decking, single studs, wide slab</td>
<td>15</td>
<td>19</td>
<td>15</td>
</tr>
<tr>
<td>W-Dek® geometry, no decking, single studs, narrow slab</td>
<td>8</td>
<td>6</td>
<td>11</td>
</tr>
<tr>
<td>W-Dek® geometry, no decking, stud pairs, wide slab</td>
<td>28</td>
<td>16</td>
<td>22</td>
</tr>
<tr>
<td>W-Dek® decking², stud pairs, wide slab</td>
<td>-4</td>
<td>-14</td>
<td>-3</td>
</tr>
<tr>
<td>W-Dek® geometry, no decking, stud pairs, narrow slab</td>
<td>26</td>
<td>7</td>
<td>25</td>
</tr>
<tr>
<td>W-Dek® decking², stud pairs, narrow slab</td>
<td>-11</td>
<td>-9</td>
<td>-8</td>
</tr>
<tr>
<td>KF70® decking, single studs, 100mm high</td>
<td>28</td>
<td>25</td>
<td>30</td>
</tr>
<tr>
<td>KF70® decking, single studs, 130mm high</td>
<td>3-24</td>
<td>7-31</td>
<td>3-22</td>
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<td>KF70® decking, stud pairs</td>
<td>-10-30</td>
<td>43-154</td>
<td>-7-22</td>
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<tr>
<td>KF70® decking, stud pairs, waveform element</td>
<td>12-18</td>
<td>-168-(-10)</td>
<td>7-10</td>
</tr>
</tbody>
</table>

$^1$: RPT: Rib punch-through, SS: Stud shearing, RS: Rib shearing, SP: Stud pull-out
$^2$: different sheeting thickness, $^3$: conventionally reinforced specimens failed before reaching 6mm slip
$^4$: specimen with ring device failed before reaching 6mm slip (due to test method)
4.5.2.9 **Waveform reinforcement element**

The push-out test results presented in Figure 4-74 show that a bottom cross-wire is needed to form an integral part of the waveform element in order to provide a proper anchorage. In the tests where no cross-wire was used, the specimens still experienced a brittle stud pull-out failure where the failure cone was such that its failure surface propagated between the two inner longitudinal wires. After failure, these specimens were cut longitudinally and the studs and sheeting could be removed from the cover slab with little effort. The brittle behaviour of the connection is also indicative of this pull-out failure. The specimens including cross-wires, on the other hand, experienced ‘typical’ rib punch-through behaviours where eventually the studs fractured at large values of slip.

The presence of the waveform reinforcement element including a transverse cross-wire increased the average maximum strength of the shear connections incorporating KF70 geometries by about 20%. At 6mm slip, these specimens generally fulfilled the ductility requirements of Eurocode 4 (CEN 2004) and provided, on average, 80%
more strength than those without the element. This makes the waveform reinforcement element a suitable measure to overcome the lack of ductility when pairs of headed studs in both internal or edge beam applications are used. From the tests described in this Chapter, it was also found that 6mm diameter bars provided sufficient resistance to reinforce against concrete pull-out or rib shearing failures. Another beneficial effect of the application of the waveform element was the reduction in variability of test results, especially at 6mm slip, since failure was no longer controlled by the concrete tensile strength (see also Ernst et al. (2004)).

**Figure 4-74:** Influence of bottom cross-wire in waveform reinforcement element for stud pairs specimens incorporating KF70\textsuperscript{®} decking

In Figure 4-75 and Figure 4-76, the behaviours of single stud shear connections, where waveform reinforcement elements were used in combination with spiral stud enhancing devices, are shown. In both applications, viz. the internal beam application and edge beam application, stud strengths and load-slip curves similar to a solid slab application were obtained with the shear connectors eventually experiencing stud shearing failures. However, in comparison to solid slab shear connections, the secondary beam specimens experienced a slightly higher slip capacity which is probably due to the reduced stiffness of the connection after the rib punch-through failure wedges had formed in front of the shear connectors (Figure 4-77).
The strain measurements in the longitudinal waveform reinforcement bars give a better insight into the behaviour of the element. For the specimens incorporating a narrow concrete slab, both the longitudinal bars next to the shear connectors (inside) and the longitudinal bars further away (outside) initially experienced strains in the rear rib of the specimen with the strains being higher in the vicinity of the stud.
connectors (Figure 4-75). The gauges positioned in the front rib experienced strains once the load-slip curve flattened i.e. cracks had developed. At that stage, the strains in the bars in the rear ribs remained constant and eventually started to drop whereas the strains in the bars in the front rib continued to increase as the cracks widened. Again, the bar in the vicinity of the shear connection experienced the majority of the vertical strains. In the wide internal beam application as illustrated in Figure 4-76, only the bars in the vicinity of the connection experienced significant strain readings during testing. Again, the bar section in the rear rib initially experienced larger amounts of strains while the section in the front rib bars started to experience increased strains in the further course of the testing. It can be concluded that for applications where the shear connections are close to a free concrete edge, all four longitudinal wires of the element are required but for wide internal beam applications, the outside bars could actually be omitted if single stud shear connections are used. However, where pairs of studs in internal beam applications were tested, the outside wires of the component also experienced significant strains (see test report Project CTR-RIB4 (Ernst and Wheeler 2006d)). Generally, the strains measured in the longitudinal bars of the element, when used in wider concrete slabs, were significantly lower than bar strains in narrow concrete slabs applications. It should be noted that from the limited number of tests where waveform strain gauge measurements were undertaken, the reason for the initial differences in bar strains in the two ribs is not clear. This requires further investigation.

After failure, the shear connection shown in Figure 4-76 (RIB4-SDM4) was cut longitudinally along its centreline (Figure 4-77). It can be seen that the concrete surrounding the shear connection was still intact and no visible horizontal concrete cracks had developed at the rear side of the concrete rib, consequently the waveform reinforcement element had successfully suppressed the onset of stud pull-out or rib shearing failures which would have been experienced otherwise. It can also be seen that the concrete had fully penetrated the spiral enhancing device and no voids in the vicinity of the shear connector had formed during the pouring process. For this specimen, a standard concrete of 32MPa specified compression strength with a maximum aggregate size of 20mm and 75mm slump was used. The concrete surrounding the shear connector had remained sound and undamaged during testing.
which resulted in low bending deformations of the shank of the stud and, hence, the full utilization of the shear connector material.

Figure 4-77: Longitudinal cut through shear connection incorporating spiral enhancing device and waveform element after failure (RIB4-SDM4)

Where stud pairs in W-Dek® shear connections were used, the combination of the spiral enhancing device and the waveform reinforcement element also provided much improved strength and ductility, for both internal and edge beam applications (Figure 4-78 and Figure 4-79). In contrast, the use of the waveform element on its own did not result in a significant increase in shear connection strength as the specimens experienced rib punch-through failures. However, much increased shear connection deformation capacities were observed in these tests. Note that the conventionally reinforced specimens shown in Figure 4-78 and Figure 4-79 included 1.00mm thick steel sheeting compared to 0.75mm thick sheeting which was used for the other specimens. The increased sheeting thickness might have increased its strength and ductility slightly (see Chapter 4.5.2.5). The use of the spiral enhancing device on its own did not result in any significant increases in strength and resulted in a more brittle behaviour. Therefore, it is advisable to only use the device in combination with the waveform component in stud pair applications.
4.5.2.10 Tensile forces in shear connectors

The strain gauge measurements of specimen RIB3-SWD9 (tested in the new horizontal push-out test rig described in Chapter 3.3) shown in Figure 4-80 are best
suited to illustrate the two different behaviours which were generally experienced. This particular specimen initially experienced a rib punch-through failure accompanied by a sharp drop in load and, at very large slips, a stud pull-out failure characterized by another drop in load. During the initial stages of loading, the tensile forces which developed in the shanks of the shear connectors (front and rear stud) increased proportionally to the lateral uplift forces measured in the restraining beam positioned at the loading side of the concrete slab (front side). However, no correlation with the concrete gauge strain measurements at the rear side of the rear concrete rib (Gauge 0 and Gauge 100L) could be observed. Hence, the vertical stud forces seemed to be largely controlled by the global force distribution of the specimen rather than by the local shear connection behaviour. It can also be seen that, during the initial stages, the vertical strains measured between the concrete rib and cover slab in the centreline of the specimen (Gauge 0) started to exceed the concrete tensile capacity. Consequently, a horizontal crack must have formed at the rear side of the concrete rib. The horizontal crack remained localized and had not yet fully propagated transversely across the concrete rib (no significant strain readings in Gauge 100L) when rib punch-through failure occurred and the load dropped. The drop in load stabilized the concrete strain measurements briefly, but further increases in slip resulted in continuous crack growths and propagation, although at a lower rate, while the load remained constant. During that period, the stud tensile forces still seemed to remain unaffected by the horizontal crack at the rear side of the concrete rib and continued to develop proportionally to the lateral uplift force at the loaded side of the concrete slab. Once the strains measured in both concrete gauges started to increase more rapidly (at around 18mm slip), i.e. an immediate widening of the crack occurred, another sharp drop in load was experienced. This would have indicated the rapid propagation of the cracks forming a stud pull-out failure cone. At that stage, both studs experienced a drop in tensile forces while the specimen restraining beam experienced a significant rise in uplift forces. Hence, the tensile forces in the shear connectors developed separately from the global lateral uplift forces experienced in the specimen, i.e. separation between the shear connections and the concrete cover slab had occurred and, hence, stud pull-out failure.
Consequently, the strain measurements in this specimen clearly prove that the stud pull-out failure, against popular belief, is not initiated by the development of high tensile forces in the shank of the stud connectors, but by failure of the concrete rib – cover slab interface. The crack is thought to initiate from the rear top corner of the concrete rib and to longitudinally propagate across the concrete rib while locally passing over the head of the shear connector. Strain gauge measurements in specimens with narrow concrete slabs indicated a similar behaviour for shear connections which experienced rib shearing failures.

In the specimens where either a pure rib punch-through failure occurred or where the propagation of this lowly inclined horizontal crack was suppressed by the application of the waveform reinforcement element, the stud forces remained proportional to the lateral uplift forces measured in the restraining beam in front of the specimen during the entire test. In the specimen shown in Figure 4-81 where the waveform reinforcement element in combination with spiral enhancing devices was provided, the tensile forces in the stud shanks were found to display behaviours similar to those observed in solid slab applications (see Chapter 4.5.1.4). However, the magnitudes of the stud tensile forces measured were slightly higher (9%-19% of the maximum shear force) which is probably an indication of the slightly higher shear deformations.
at the base of the stud connectors due to the formation of rib punch-through cracks. The increase in stud tensile forces might also be a possible explanation for the slightly reduced stud shearing capacity of the connectors compared to solid slab applications. However, in relation to the ultimate tensile capacity of the shear connectors, the forces in the stud shanks still remained low (9%-18% of $T_u$).

![Figure 4-81: Uplift force measurements compared to load-slip behaviour for a specimen incorporating spiral devices and waveform element (RIB4-SWD4)](image)

The stud uplift tensile forces in the various shear connections measured at maximum load are summarized in Table 4-27. It can be seen that, with one exception, the tensile forces experienced typically amounted to 10%-30% of the stud shear forces regardless of the shear connection layout and failure mode experienced. This consistency is another indication that the lateral forces experienced in the connectors reflected the global boundary conditions of the specimen rather than the particular local shear connection behaviour and were not the cause for the premature stud pull-out or rib shearing failures. The conventionally reinforced specimen with an eccentric slab layout (RIB5-SDM3) experienced an unusually high shear connection force in one of the connectors along with an unusual ductile behaviour of the shear connection compared to similar specimens. The reason for this behaviour is not understood and this individual result should probably not be overrated.
Table 4-27: Tensile forces measured in the shank of the shear connectors

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Stud tensile force at maximum load $T_{stud}$</th>
<th>$T_{stud} / P_{max}$</th>
<th>Failure mode$^1$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>[kN]</td>
<td>[%]</td>
<td></td>
</tr>
<tr>
<td>Front studs</td>
<td>Back studs</td>
<td>Front studs</td>
<td>Back studs</td>
</tr>
<tr>
<td>No decking, single stud, conventionally reinforced, wide slab</td>
<td>-</td>
<td>18-20</td>
<td>-</td>
</tr>
<tr>
<td>No decking, single stud, conventionally reinforced, narrow slab</td>
<td>16</td>
<td>16</td>
<td>21</td>
</tr>
<tr>
<td>No decking, single stud, spiral enhancing device, wide concrete slab</td>
<td>24</td>
<td>23</td>
<td>22</td>
</tr>
<tr>
<td>No decking, single stud, spiral enhancing device, narrow slab</td>
<td>27</td>
<td>-</td>
<td>33</td>
</tr>
<tr>
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<td>15</td>
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<td>13</td>
<td>26</td>
<td>9</td>
</tr>
<tr>
<td>wide slab</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Steel decking, single stud, spiral enhancing device, waveform element,</td>
<td>15</td>
<td>-</td>
<td>12</td>
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<tr>
<td>narrow slab</td>
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<td></td>
<td></td>
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<td>No decking, stud pairs, conventionally reinforced, narrow slab</td>
<td>6-14</td>
<td>7-13</td>
<td>11-27</td>
</tr>
<tr>
<td>No decking, stud pairs, spiral enhancing device, wide slab</td>
<td>11-12</td>
<td>18</td>
<td>16-18</td>
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<td>16-19</td>
</tr>
<tr>
<td>Steel decking, stud pairs, conventionally reinforced, eccentric slab</td>
<td>-</td>
<td>23-44</td>
<td>-</td>
</tr>
<tr>
<td>Steel decking, stud pair, spiral enhancing device, waveform element,</td>
<td>-</td>
<td>16-23</td>
<td>-</td>
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<tr>
<td>wide slab</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>Steel decking, stud pair, spiral enhancing device, waveform element,</td>
<td>-</td>
<td>12-25</td>
<td>-</td>
</tr>
<tr>
<td>eccentric slab</td>
<td></td>
<td></td>
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</tr>
</tbody>
</table>

$^1$: RPT: Rib punch-through, SS: Stud shearing, RS: Rib shearing, SP: Stud pull-out

### 4.6 Summary

The application of the two Australian types of trapezoidal steel decking geometries was found to result in a significant reduction in shear connection strength and a lack of ductility due to the occurrence of premature concrete-related failure modes. In all of these concrete-related failure modes, the concrete tensile strength generally had a big influence on the shear connection strength. Most of the conventionally reinforced specimens experienced rib punch-through types of failures which were found to be
influenced by the position of the shear connectors in the concrete rib, the thickness of the steel sheeting and the number of shear connectors per concrete rib. The rib punch-through failures were typically accompanied by longitudinal rib splitting failures which further weakened the shear connection. The application of transverse reinforcement bars in the concrete rib can limit the impact of this failure mode and increase the shear connection strength.

However, the use of stud pairs in shear connections can create other brittle failure modes such as rib shearing or stud pull-out failures. For these cases, an increase in steel decking thickness, a more favourable stud position or a horizontal bottom layer of cover slab reinforcement could not provide any significant improvements of the shear connection behaviour. From strain measurements, it was found that both of these failures modes are characterized by a near-horizontal concrete crack which initially forms between concrete rib and cover part of the slab at the rear side of the shear connection and eventually propagates over the head of the studs. The use of waveform reinforcement elements, in which transverse bottom reinforcement bars need to form an integral part, proved to be a suitable measure to control these cracks and prevent the unsatisfactory shear connection behaviours.

The application of stud performance-enhancing devices was found to increase the shear connection resistance against rib punch-through failure by around 10%-30% by confining the concrete around the base of the shear connector and increasing the potential failure surface. However, this strength increase also seemed to trigger subsequent failure modes of stud pull-out in wide concrete slabs or rib-shearing in narrow concrete slabs. Shear connections including the addition of the waveform reinforcement element to suppress the effects of these subsequent types of failures generally displayed behaviours very similar to shear connection behaviours in solid slab applications and provided sufficient ductility. The respective test specimens were able to reach full solid-slab strength in an internal single-stud application, and 90% of this strength in edge beam applications with a slab outstand of less than 200 mm. The strength for stud pairs was also significantly increased to 77% of the solid-slab strength in wide specimens, and to 68% in narrow specimens (600 mm wide). The minimum ductility requirement of Eurocode 4 (CEN 2004) was generally satisfied for these applications.
The concrete slab width of the test specimen was found to have a profound impact on the shear connection strength and failure mode. Wide slabs generally provided a higher strength than narrow specimens. This influence was more evident when pairs of shear connectors were tested. For the W-Dek® decking geometry, it was found that the concrete slabs of test specimens should be at least 1800 mm wide for the geometry of internal composite beam applications to be correctly simulated.
5. PUSH-OUT TESTS - PRIMARY BEAM APPLICATION

5.1 Introduction

In primary beam applications where the steel decking runs parallel to the steel section, a concrete haunch is created. As typically no reinforcement is provided in these haunches, numerous premature concrete related failure modes might be experienced (see Chapter 2.3.3). These failures are known to reduce the shear connection strength significantly and can be of a very brittle nature. The use of haunch reinforcement similar to the types provided in haunches of solid slabs (see Chapter 2.2.3) might be a suitable measure to suppress these types of failures and increase the strength and ductility of the shear connection.

The push-out tests, presented and discussed in the following, had the aim to investigate the behaviour of the haunches created by the Australian types of trapezoidal steel decking. Its objectives were to identify the possible applications where brittle shear connection behaviours could occur and to provide possible reinforcing solutions which ensure sufficiently ductile behaviour. During the course of testing, two types of reinforcing components that control the concrete-related failure modes were investigated: one being a straight ladder reinforcement placed in the haunch at the base of the shear connection; and one being a stirrup or handle-bar reinforcement tying the concrete haunch to the concrete cover slab. Versions of these types of reinforcing components are currently commercially available as HAUNCHMESH™ shear reinforcement from OneSteel.
The various push-out tests and their results are presented in Chapters 5.2.1 and 5.3.1 for KF70® steel decking applications and in Chapter 5.2.2 and 5.3.2 for W-Dek® steel decking applications. A description of the failure modes observed is given in Chapter 5.4. More information about the primary beam push-out tests can be found in the accompanying test reports (Ernst and Wheeler 2006e; f; g; h; i). The test results are assessed and analysed in Chapter 5.5, also taking into account further test results published elsewhere and, based on these findings, preliminary suggestions for the shear connection design of primary beam applications incorporating the Australian types of trapezoidal steel decking are made in Chapter 5.6.

5.2 Test programs and specimens

5.2.1 Shear connection in KF70® geometry

5.2.1.1 General

A total of 15 push-out tests on the KF70® steel decking geometry in 5 different test series were undertaken. The tests of series SRG-P01 to SRG-P03 and OSR-DM01 were performed using the one-sided vertical test rig as described in Chapter 3.2. The limitations described therein for this test rig apply. The tests of series OBD-HS01 were performed on the new horizontal test rig as described in Chapter 3.3. The specimens were generally 600mm wide and 130mm high. For the specimens consisting of four 19mm diameter stud shear connectors, the slabs were 600mm long while for the specimens consisting of six 19mm diameter stud connectors, they were 800mm long. The shear connectors were automatically fast-welded either directly to a 20mm thick and 100mm wide flange plate in the specimens where a concrete haunch was formed with the steel decking split, or through the steel decking to the flange plate in the specimens where the steel decking remained unsplit. The studs were generally placed in the centreline of the haunch at the designated longitudinal spacings between individual connectors. In the longitudinal direction, the specimens
were reinforced using a top layer of four 10mm diameter longitudinal reinforcement bars at 150mm transverse spacings which were placed 20mm clear of the concrete slab top face. The transverse reinforcement was designed to fulfil the longitudinal Type 1, 2 and 3 shear reinforcement requirements given in Section 9 of AS2327.1 (Standards Australia 2003a) and was placed directly on top of the steel decking. A typical specimen containing no additional haunch reinforcement is shown in Figure 5-1.

![Figure 5-1: Typical push-out test specimen without haunch reinforcement](image)

Where straight ladder haunch reinforcement was used, the component was typically cut from a RL1218 mesh (six 11.9mm diameter transverse wires at 100mm spacings and two 7.6mm diameter longitudinal wires at 200mm spacings) and placed at mid-height of the concrete haunch (see Figure 5-2). However, in specimen SRG-P03-PR3 where the steel decking was unsplit, the component needed to be slightly modified as the haunch was narrower, hence the longitudinal wires were provided at a closer transverse spacing (150mm crs) and the transverse wires were shortened.

For the earlier specimen (SRG-P02-P2), the vertical haunch reinforcement was of a stirrup type which consisted of loops of 6mm diameter round bars bent around 12mm pins and welded to longitudinal wires at 100mm spacings between individual loops. The component was 170mm wide and 100mm high and simply placed in the haunch of the specimen (Figure 5-3). In the later specimens (OBD-HS01 series), a handle-bar type of reinforcement which was basically a bent 10mm diameter reinforcement bar of steel grade 500N with an overall height of 80mm was used as vertical haunch reinforcement (Figure 5-4). The component was welded to the longitudinal slab reinforcement from where it extended into the concrete haunch.
In some of the early specimens, ring enhancing devices which were of the same type as used in secondary beam applications (see Chapter 4.2.1.1) were clipped into position over the heads of the studs prior pouring.
The specimens of each individual test series were cast in the horizontal position from the same batch of concrete together with numerous 150x100mm concrete cylinders which were stored alongside the specimen and tested at regular intervals.

5.2.1.2 Haunches with split steel decking

In applications of relatively wide concrete haunches, i.e. where the steel decking is split, the differences in behaviour of unreinforced concrete haunches, haunches containing straight ladder reinforcement and haunches including additional vertical haunch reinforcement were investigated. Additionally, some of the specimens included the ring enhancing device in order to investigate possible benefits for primary beam shear connections. The longitudinal spacing of the headed studs was set to 100mm which fulfilled the minimum spacing requirement of AS2327.1 (Standards Australia 2003a), being more than 5 times the stud diameter.

<table>
<thead>
<tr>
<th>Code</th>
<th>Studs (height in mm)</th>
<th>Sheet thickness (mm)</th>
<th>Haunch width $b_h$ (mm)</th>
<th>Long. stud spacing (mm)</th>
<th>Ladder haunch reinforcement</th>
<th>Vertical haunch reinforcement</th>
<th>Ring device</th>
<th>Target concrete strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SRG-P01-P1</td>
<td>4x Singles (100)</td>
<td>0.75</td>
<td>220</td>
<td>100</td>
<td>No</td>
<td>No</td>
<td>No</td>
<td>30</td>
</tr>
<tr>
<td>SRG-P01-P2</td>
<td>4x Singles (100)</td>
<td>0.75</td>
<td>220</td>
<td>100</td>
<td>No</td>
<td>No</td>
<td>Yes</td>
<td>30</td>
</tr>
<tr>
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<td>4x Singles (100)</td>
<td>0.75</td>
<td>220</td>
<td>100</td>
<td>Yes</td>
<td>No</td>
<td>No</td>
<td>40</td>
</tr>
<tr>
<td>SRG-P02-P2</td>
<td>4x Singles (100)</td>
<td>0.75</td>
<td>220</td>
<td>100</td>
<td>Yes</td>
<td>Yes</td>
<td>No</td>
<td>40</td>
</tr>
<tr>
<td>SRG-P02-P3</td>
<td>4x Singles (100)</td>
<td>0.75</td>
<td>220</td>
<td>100</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
<td>40</td>
</tr>
<tr>
<td>SRG-P02-P4</td>
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<td>0.75</td>
<td>220</td>
<td>100</td>
<td>No</td>
<td>No</td>
<td>No</td>
<td>40</td>
</tr>
</tbody>
</table>

1: measured at bottom of haunch
2: strip of RL1218 mesh (Ø11.9mm transv. wires @100mm crs, Ø7.6mm long. wires @200mm crs)
3: five stirrup components @100mm crs (Ø6mm welded loops, 100mm high and 170mm wide)
5.2.1.3 Haunches with unsplit steel decking

Where the steel decking was unsplit, the haunch width is directly defined by the decking geometry. For this type of application, the use of straight and vertical haunch reinforcement was also investigated whereby a particular emphasis was placed on the longitudinal anchorage of the transverse haunch reinforcement. Therefore, the straight longitudinal haunch reinforcement in series OBD-HS01, consisting of two 10mm diameter reinforcement bars, was either welded or tied to the handle-bar components or, in another case, completely omitted. The influences of the concrete strength and the stud connector height on the behaviour of unreinforced concrete haunches were other factors that were investigated.

Table 5-2: Test specimens with unsplit KF70® steel decking

<table>
<thead>
<tr>
<th>Code</th>
<th>Studs (height in mm)</th>
<th>Sheet thickness (mm)</th>
<th>Haunch width (b_h) (mm)</th>
<th>Long. stud spacing (mm)</th>
<th>Ladder haunch reinforcement</th>
<th>Vertical haunch reinforcement</th>
<th>Target concrete strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SRG-P03-PR1</td>
<td>4x Singles (100)</td>
<td>1.00</td>
<td>136</td>
<td>100</td>
<td>Yes(^2)</td>
<td>No</td>
<td>40</td>
</tr>
<tr>
<td>SRG-P03-PR2</td>
<td>4x Singles (130)</td>
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<td>136</td>
<td>100</td>
<td>No</td>
<td>No</td>
<td>40</td>
</tr>
<tr>
<td>OSR-DM01-P01</td>
<td>4x Singles (100)</td>
<td>1.00</td>
<td>136</td>
<td>100</td>
<td>No</td>
<td>No</td>
<td>30</td>
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<tr>
<td>OSR-DM01-P02</td>
<td>4x Singles (100)</td>
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<td>100</td>
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<tr>
<td>OSR-DM01-P03</td>
<td>4x Singles (100)</td>
<td>1.00</td>
<td>136</td>
<td>100</td>
<td>Yes(^4), no long. bars in haunch</td>
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<td>40</td>
</tr>
<tr>
<td>OBD-HS01-P01</td>
<td>6x Singles (120)</td>
<td>0.75</td>
<td>136</td>
<td>100</td>
<td>Yes(^4), long. bars welded to transv. reo</td>
<td>Yes</td>
<td>40</td>
</tr>
<tr>
<td>OBD-HS01-P02</td>
<td>6x Singles (120)</td>
<td>0.75</td>
<td>136</td>
<td>100</td>
<td>Yes(^4), long. bars tied to transv reo</td>
<td>Yes</td>
<td>40</td>
</tr>
<tr>
<td>OBD-HS01-P03</td>
<td>6x Singles (120)</td>
<td>0.75</td>
<td>136</td>
<td>100</td>
<td>Yes(^4), no long. bars in haunch</td>
<td>Yes</td>
<td>40</td>
</tr>
<tr>
<td>OBD-HS01-P04</td>
<td>6x Singles (120)</td>
<td>0.75</td>
<td>136</td>
<td>100</td>
<td>No</td>
<td>No</td>
<td>40</td>
</tr>
</tbody>
</table>

\(^1\): measured at bottom of haunch
\(^2\): ∅11.9mm transv. wires @100mm crs, ∅7.6mm long. wires @150mm crs
\(^3\): 8x handlebar component @100mm crs (see Figure 5-4)
\(^4\): ladder reinforcement is integral bottom part of handle-bar component

In order to obtain a more thorough understanding of the mechanism of the haunch reinforcement, strain gauge measurements on some of the longitudinal and transverse bars in the concrete haunch were undertaken in specimens HS01-P01 to P03. The
handle-bar strain gauges were glued to the base of the component as indicatively shown in Figure 5-4 for specimen HS01-P03.

### 5.2.2 Shear connection in W-Dek® geometry

A total of ten push-out-tests were carried out on the Lysaght W-Dek® geometry in primary beam applications in order to optimize the design of the haunch reinforcing components. All specimens consisted of an element of slab which was 900mm wide, 900mm long and 140mm high that was connected to a steel flange plate of 200mm width and 20mm thickness. Headed studs of 19mm diameter and 115mm height after welding were automatically fast-welded onto the steel plate whereby the number and spacings of the studs were varied in accordance with Table 5-3. Where single studs were used, the studs were placed in the centreline of the concrete haunch at the designated longitudinal spacings. The transverse spacing for stud pairs was generally set to 60mm between the centreline of the adjacent studs fulfilling the minimum requirement for transverse stud spacing of AS2327.1 (Standards Australia 2003a). The concrete haunches of the specimens were 240mm wide including the two adjacent lap joints of the steel decking. The split steel deckings were pin-fixed to the steel plate before concreting. Additionally, a solid slab specimen with an otherwise similar shear connection lay-out was included as a reference specimen in this investigation. The tests of the LST-DEC1 series were performed using the single-sided push-out test rig as described in Chapter 3.2 whereas the comparison specimen of the CTR-RIB5 series was tested on the horizontal push-out test rig described in Chapter 3.3.

For the specimens which included the ladder haunch reinforcement, welded reinforcement meshes were fabricated with their components and dimensions being variables in the tests as shown in Table 5-3. The optimal position of the ladder reinforcement, be it on top of the lap joint or closer to mid-height of the concrete haunch, also formed part of this investigation. The vertical haunch reinforcement consisted of handle-bar components bent from reinforcement steel of grade 500N with the standard geometric properties given in Figure 5-5. The diameter, overall height and number of handle-bar components were varied in accordance with Table
5-3. The handle-bar reinforcement was generally welded to the bottom of the longitudinal bars of the ladder haunch reinforcement and tied to the longitudinal top face reinforcement.

**Table 5-3**: Test specimens with split W-Dek® steel decking

<table>
<thead>
<tr>
<th>Code</th>
<th>Studs (height in mm)</th>
<th>Studs thickness (mm)</th>
<th>Haunch width $b_h^1$ (mm)</th>
<th>Long. stud spacing (mm)</th>
<th>Ladder haunch reinforcement</th>
<th>Vertical haunch reinforcement $^3$</th>
</tr>
</thead>
<tbody>
<tr>
<td>LST-DEC1-HM01</td>
<td>6x Singles (115)</td>
<td>0.75</td>
<td>240</td>
<td>100</td>
<td>Transv: 909.5@100crs Long: 207.6@150crs</td>
<td>209.5@300crs, 95mm high, lap joint</td>
</tr>
<tr>
<td>LST-DEC1-HM02</td>
<td>6x Singles (115)</td>
<td>0.75</td>
<td>240</td>
<td>100</td>
<td>Transv: 909.5@100crs Long: 207.6@150crs</td>
<td>209.5@300crs, 80mm high, 35mm clear from base</td>
</tr>
<tr>
<td>LST-DEC1-HM03</td>
<td>6x Singles (115)</td>
<td>0.75</td>
<td>240</td>
<td>100</td>
<td>Transv: 909.5@100crs Long: 207.6@150crs</td>
<td>209.5@300crs, 80mm high, lap joint</td>
</tr>
<tr>
<td>LST-DEC1-HM04</td>
<td>4x Singles (115)</td>
<td>0.75</td>
<td>240</td>
<td>150</td>
<td>Transv: 609.5@150crs Long: 207.6@150crs</td>
<td>No</td>
</tr>
<tr>
<td>LST-DEC1-HM05</td>
<td>4x Pairs$^2$ (115)</td>
<td>0.75</td>
<td>240</td>
<td>150</td>
<td>Transv: 609.5@150crs Long: 207.6@150crs</td>
<td>509.5@150crs, 95mm high, lap joint</td>
</tr>
<tr>
<td>LST-DEC1-HM06</td>
<td>4x Pairs$^2$ (115)</td>
<td>0.75</td>
<td>240</td>
<td>150</td>
<td>Transv: 6012@150crs Long: 209.5@150crs</td>
<td>4012@150crs, 95mm high, lap joint</td>
</tr>
<tr>
<td>LST-DEC1-HM07</td>
<td>4x Pairs$^2$ (115)</td>
<td>0.75</td>
<td>240</td>
<td>150</td>
<td>Transv: 6012@150crs Long: 209.5@150crs</td>
<td>4012@150crs, 80mm high, 35mm clear from base</td>
</tr>
<tr>
<td>LST-DEC1-HM08</td>
<td>4x Pairs$^2$ (115)</td>
<td>0.75</td>
<td>240</td>
<td>100</td>
<td>Transv: 8012@100crs Long: 209.5@150crs</td>
<td>4012@100crs, 95mm high, lap joint</td>
</tr>
<tr>
<td>LST-DEC1-HM09</td>
<td>6x Singles (115)</td>
<td>Solid Slab</td>
<td>N/A</td>
<td>100</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>LST-DEC1-HM10</td>
<td>6x Singles (115)</td>
<td>0.75</td>
<td>240</td>
<td>100</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>CTR-RIB5-SDM7</td>
<td>6x Singles (115)</td>
<td>0.75</td>
<td>240</td>
<td>100</td>
<td>Transv: 909.5@100crs Long: 207.6@150crs</td>
<td>209.5@300crs, 95mm high, lap joint</td>
</tr>
</tbody>
</table>

$^1$: measured at bottom of haunch, $^2$: transverse stud spacing: 60mm crs

$^3$: handlebar component (see Figure 5-5)

Figure 5-5: Handlebar reinforcement component for W-Dek® geometry

The transverse top face reinforcement for each specimen was determined in accordance with Section 9 of AS 2327.1 (Standards Australia 2003a) to ensure sufficient Type 1 longitudinal shear transfer in the concrete. It was bent down on
both ends to prevent a horizontal concrete failure of the transverse edges of the narrow concrete slab. The longitudinal top face reinforcement consisted of six 10mm diameter reinforcement bars welded to the transverse bars. The top mesh was positioned to provide 25mm clear concrete cover to the slab top surface. For the specimens without the vertical haunch reinforcement component, a bottom mesh of welded reinforcement bars placed on top of the steel decking was additionally provided to fulfil the requirements of Type 3 longitudinal shear reinforcement as given in AS2327.1 (Standards Australia 2003a). In Figure 5-6, a test specimen including ladder and handle-bar haunch reinforcement is indicatively shown.

![Diagram of reinforcement details](image)

**Figure 5-6:** Test specimen including ladder and vertical haunch reinforcement

![Image of test specimen](image)

**Figure 5-7:** Position of haunch reinforcement strain gauges in specimen DEC1-HM01
Similar to the investigations conducted on the KF70® steel decking geometry, some of the specimens were equipped with strain gauges fixed to the transverse and longitudinal bars of the straight ladder reinforcement provided in the concrete haunch as shown in Figure 5-7 for specimen DEC1-HM01.

The test specimens were poured in the horizontal position from the same batch of concrete for each series. Alongside the test specimens, concrete cylinders were cured to determine the concrete compressive strength at the necessary ages. All of the test specimens and their companion concrete cylinders were kept moist until they were prepared for testing.

5.3 Test results and observations

5.3.1 Shear connection in KF70® geometry

5.3.1.2 General

The material properties of the steel decking and shear connectors were given earlier in Table 4-4. Once a specimen reached its designated concrete target strength, its formwork was removed and the steel flange plate of the specimen was attached to the push-out test rig. The end loading pattern was designed to only load the cover slab in the side regions of the specimens (see also Figure 3-12). The cover part above the concrete haunch typically remained unloaded, i.e. a recess of about 200mm width was formed.

The testing was generally displacement controlled while the speed varied during testing. Each specimen was cycled in a similar fashion as the secondary beam specimens described earlier, i.e. at least three times between 10 kN and a proof load of about 40% of the expected maximum shear strength. After the last cycle was completed, the slip was applied continuously until stud failure occurred or until the load had dropped a significant amount below its maximum.
The maximum shear strength measured before the load started to drop, the measured concrete strengths, the failure modes experienced and, as an indication of the ductility of the specimen, its strength at 6 mm slip are summarized for the different applications in Table 5-4 and Table 5-5. The load-slip curves for the tests are given in Figure 5-8 and Figure 5-9.

5.3.1.3 Haunches with split steel decking

<table>
<thead>
<tr>
<th>Code</th>
<th>Ladder haunch reinforcement</th>
<th>Vertical haunch reinforcement</th>
<th>Ring device</th>
<th>Measured concrete strength (MPa)</th>
<th>Failure mode</th>
<th>Max stud strength $P_{\text{max}}$ (kN)</th>
<th>Stud strength at 6mm slip $P_{\text{ass}}$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SRG-P01-P1</td>
<td>No</td>
<td>No</td>
<td>No</td>
<td>26.5</td>
<td>LS</td>
<td>66</td>
<td>64</td>
</tr>
<tr>
<td>SRG-P01-P2</td>
<td>No</td>
<td>No</td>
<td>Yes</td>
<td>26.0</td>
<td>LS</td>
<td>74</td>
<td>59</td>
</tr>
<tr>
<td>SRG-P02-P1</td>
<td>Yes</td>
<td>No</td>
<td>No</td>
<td>38.5</td>
<td>HS</td>
<td>108</td>
<td>99</td>
</tr>
<tr>
<td>SRG-P02-P2</td>
<td>Yes</td>
<td>Yes</td>
<td>No</td>
<td>41.0</td>
<td>$SE$</td>
<td>121</td>
<td>121</td>
</tr>
<tr>
<td>SRG-P02-P3</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
<td>41.0</td>
<td>HS</td>
<td>126</td>
<td>98</td>
</tr>
<tr>
<td>SRG-P02-P4</td>
<td>No</td>
<td>No</td>
<td>No</td>
<td>39.5</td>
<td>LS/HS</td>
<td>109</td>
<td>48</td>
</tr>
</tbody>
</table>

1: LS: Longitudinal splitting, HS: Haunch shearing, SS: Stud shearing, SE: Splitting of slab edges

Observations:
- The unreinforced concrete haunches were found to experience either longitudinal splitting or a combination of longitudinal splitting and haunch shearing failure and were characterized by either a low shear connection strength or little ductility (P01-P1 and P02-P4).
- The use of straight ladder reinforcement in concrete haunches was able to suppress longitudinal splitting failure but the specimen still experienced relatively brittle haunch shearing failure (P02-P1).
- The use of the ring enhancing device improved the shear connection strength but did not alter the failure mode (P01-P1/P01-P2 and P02-P1/P02-P3).
- The use of vertical haunch reinforcement in combination with straight ladder reinforcement was able to increase shear connection strength and ductility up to a satisfactory level (P02-P2). An even better shear connection behaviour
might have been possible but the specimen failed prematurely due to splitting cracks forming at the transverse edges of the narrow concrete slab, a failure not expected in wider concrete slab applications.

**Figure 5-8:** Load-slip behaviour of SRG-P01 specimens

**Figure 5-9:** Load-slip behaviour of SRG-P02 specimens
5.3.1.4 Haunches with unsplit decking

Table 5-5: Test results of specimens with unsplit KF70® steel decking

<table>
<thead>
<tr>
<th>Code</th>
<th>Stud height (mm)</th>
<th>Ladder haunch reinforcement</th>
<th>Vertical haunch reinforcement</th>
<th>Measured concrete strength (MPa)</th>
<th>Failure mode</th>
<th>Max stud strength $P_{\text{max}}$ (kN)</th>
<th>Stud strength at 6mm slip $P_{\text{6mm}}$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SRG-P03-PR1</td>
<td>100</td>
<td>Yes</td>
<td>No</td>
<td>40</td>
<td>HS</td>
<td>85.8</td>
<td>75</td>
</tr>
<tr>
<td>SRG-P03-PR2</td>
<td>130</td>
<td>No</td>
<td>No</td>
<td>40</td>
<td>HS/SE</td>
<td>76.3</td>
<td>68</td>
</tr>
<tr>
<td>OSR-DM01-P01</td>
<td>100</td>
<td>No</td>
<td>No</td>
<td>30</td>
<td>HS</td>
<td>74</td>
<td>46</td>
</tr>
<tr>
<td>OSR-DM01-P02</td>
<td>100</td>
<td>No</td>
<td>No</td>
<td>33</td>
<td>HS</td>
<td>69</td>
<td>30</td>
</tr>
<tr>
<td>OSR-DM01-P03</td>
<td>100</td>
<td>No</td>
<td>No</td>
<td>41</td>
<td>HS</td>
<td>86</td>
<td>37</td>
</tr>
<tr>
<td>OBD-HS01-P01</td>
<td>120</td>
<td>No</td>
<td>Yes, long. bars welded to transv. reo</td>
<td>$\approx 50$²</td>
<td>SS</td>
<td>130</td>
<td>129</td>
</tr>
<tr>
<td>OBD-HS01-P02</td>
<td>120</td>
<td>No</td>
<td>Yes, long. bars tied to transv reo</td>
<td>$\approx 50$²</td>
<td>SS³</td>
<td>108</td>
<td>103</td>
</tr>
<tr>
<td>OBD-HS01-P03</td>
<td>120</td>
<td>No</td>
<td>Yes, no long. bars in haunch</td>
<td>$\approx 50$²</td>
<td>-⁴</td>
<td>128</td>
<td>126</td>
</tr>
<tr>
<td>OBD-HS01-P04</td>
<td>120</td>
<td>No</td>
<td>No</td>
<td>$\approx 50$²</td>
<td>LS/HS</td>
<td>115</td>
<td>0</td>
</tr>
</tbody>
</table>

¹: LS: Longitudinal splitting, HS: Haunch shearing, SS: Stud shearing, SE: Splitting of slab edges
²: specimens tested after storage >365d. Last concrete cylinder tests available at 50d.
³: one stud with faulty stud weld, i.e. weld had not completely penetrated stud.
⁴: Test stopped at 19mm slip without visible failure having occurred.

Observations:

- The specimens with unreinforced haunches predominantly experienced haunch shearing failure and behaved in a brittle manner (P03-PR2, DM01-P01 to P03 and HS01-P04). The use of higher stud connectors (P03-PR2) could not prevent this type of failure.
- The concrete strength appeared to have an effect on the shear connection strength in the specimens with unreinforced haunches but did not alter the general behaviour (compare DM01-P03 with DM01-P01).
- The use of ladder reinforcement in the concrete haunch did not have much effect on the shear connection behaviour, particularly the strength, where haunch shearing failure was experienced (P03-PR1).
Shear connection behaviour similar to solid slab behaviour was experienced where the handle-bar component was provided in the concrete haunch (HS01-P01 to P03) such that no significant effects of the different longitudinal anchorages of the transverse haunch reinforcement bars could be observed. Note that specimen HS01-P02 included a shear connector with a faulty weld which became visible after the studs had sheared off. This faulty connection is assumed to largely account for the reduced overall strength.

**Figure 5-10:** Load-slip behaviour of SRG-P03 and OSR-DM01 specimens

**Figure 5-11:** Load-slip behaviour of OBD-HS01 specimens
5.3.2 Shear connection in W-Dek® geometry

5.3.2.2 General

The properties of the steel decking and the headed studs were measured in separate tensile tests in accordance with AS1391 (Standards Australia 2005). The material properties are summarized in Table 5-6.

<table>
<thead>
<tr>
<th>Material</th>
<th>Yield strength $f_y$ (MPa)</th>
<th>Ultimate strength $f_u$ (MPa)</th>
<th>Elastic Modulus $E$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>W-Dek® steel decking, t=0.75mm</td>
<td>626</td>
<td>629</td>
<td>200000</td>
</tr>
<tr>
<td>Studs, 115mm high</td>
<td>-</td>
<td>396</td>
<td>-</td>
</tr>
</tbody>
</table>

Each specimen was stripped of its formwork once the desired concrete strength was reached and attached to the push-out test rig. The end loading pattern was designed to only load the cover slab and a recess above the concrete haunch similar to the KF70® geometry tests was formed.

The test procedure was the same as for the KF70® specimens: After cycling the specimens three times at a rate of at least 4 minutes per cycle, the slip was applied continuously until stud failure occurred or until the load had dropped a significant amount below the maximum strength. The test results are summarized in Table 5-7 and the load-slip behaviours are shown in Figure 5-12 and Figure 5-13.

5.3.2.3 Haunches with unsplit decking

Observations:
- The single stud specimen with the unreinforced concrete haunch experienced a combined haunch shearing and longitudinal splitting failure (DEC1-HM10).
- Where the single stud spacing was increased and straight ladder reinforcement was provided (DEC1-HM04), a strength similar to the solid
slab application (DEC1-HM09) could be observed and stud shearing failure occurred.

- Most of the specimens where vertical haunch reinforcement was provided, in particular all of the stud pair specimens, showed a failure of the concrete cover slab where some concrete pieces spalled from the slab (see Chapters 3.2.3 and 3.3.4). This failure is not a shear connection failure and is likely to have been introduced by the test method. Therefore, as the cover slab failure led to similar behaviour of all stud pair specimens (DEC1-HM05 to HM08), no conclusions about the influence of the haunch reinforcement placements and dimensions can be made.

- For the single stud specimens where the cover slab failure was not as severe (DEC1-HM02 and HM03), ductile shear connection behaviours with increased stud strengths compared to the unreinforced haunch application (DEC1-HM10) were experienced.

![Figure 5-12: Load-slip behaviour of single stud specimens](image-url)
**Figure 5-13**: Load-slip behaviour of stud pair specimens

**Table 5-7**: Test results of specimens with split W-Dek® steel decking

<table>
<thead>
<tr>
<th>Code</th>
<th>Studs</th>
<th>Ladder haunch reinforcement</th>
<th>Vertical haunch reinforcement</th>
<th>Measured concrete strength (MPa)</th>
<th>Failure mode</th>
<th>Max stud strength $P_{\text{max}}$ (kN)</th>
<th>Stud strength at 6mm slip $P_{6\text{mm}}$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>LST-DEC1-HM01</td>
<td>Singles</td>
<td>Yes$^2$</td>
<td>209.5 @300crs, 95mm high, lap joint</td>
<td>25.9</td>
<td>CS</td>
<td>84</td>
<td>0</td>
</tr>
<tr>
<td>LST-DEC1-HM02</td>
<td>Singles</td>
<td>Yes$^2$</td>
<td>209.5 @300crs, 80mm high, 35mm clear</td>
<td>28.8</td>
<td>CS?</td>
<td>91</td>
<td>90</td>
</tr>
<tr>
<td>LST-DEC1-HM03</td>
<td>Singles</td>
<td>Yes$^2$</td>
<td>209.5 @300crs, 80mm high, lap joint</td>
<td>28.8</td>
<td>CS?</td>
<td>96</td>
<td>96</td>
</tr>
<tr>
<td>LST-DEC1-HM04</td>
<td>Singles</td>
<td>Yes$^3$</td>
<td>No</td>
<td>31.0</td>
<td>SS</td>
<td>123</td>
<td>109</td>
</tr>
<tr>
<td>LST-DEC1-HM05</td>
<td>Pairs</td>
<td>Yes$^3$</td>
<td>509.5 @150crs, 95mm high, lap joint</td>
<td>29.6</td>
<td>CS</td>
<td>86</td>
<td>81</td>
</tr>
<tr>
<td>LST-DEC1-HM06</td>
<td>Pairs</td>
<td>Yes$^4$</td>
<td>4012 @150crs, 95mm high, lap joint</td>
<td>30.3</td>
<td>CS</td>
<td>84</td>
<td>77</td>
</tr>
<tr>
<td>LST-DEC1-HM07</td>
<td>Pairs</td>
<td>Yes$^4$</td>
<td>4012 @150crs, 80mm high, 35mm clear</td>
<td>30.3</td>
<td>CS</td>
<td>83</td>
<td>76</td>
</tr>
<tr>
<td>LST-DEC1-HM08</td>
<td>Pairs</td>
<td>Yes$^5$</td>
<td>4012 @150crs, 95mm high, lap joint</td>
<td>30.3</td>
<td>CS</td>
<td>88</td>
<td>84</td>
</tr>
<tr>
<td>LST-DEC1-HM09</td>
<td>Singles</td>
<td>N/A</td>
<td>N/A</td>
<td>31.5</td>
<td>SS</td>
<td>110</td>
<td>103</td>
</tr>
<tr>
<td>LST-DEC1-HM10</td>
<td>Singles</td>
<td>No</td>
<td>No</td>
<td>29.6</td>
<td>LS/HS</td>
<td>88</td>
<td>82</td>
</tr>
<tr>
<td>CTR-RIB5-SDM7</td>
<td>Singles</td>
<td>Yes$^2$</td>
<td>209.5 @300crs, 95mm high, lap joint</td>
<td>30.7</td>
<td>CS</td>
<td>89</td>
<td>82</td>
</tr>
</tbody>
</table>

$^1$: LS: Longitudinal splitting, HS: Haunch shearing, SS: Stud shearing, CS: Cover slab failure
$^2$: transv: 909.5 @100crs, long: 207.6 @150crs
$^3$: transv: 609.5 @150crs, long: 207.6 @150crs
$^4$: transv: 6012 @150crs, long: 209.5 @150crs
$^5$: transv: 8012 @100crs, long: 209.5 @150crs
5.4 Failure modes

5.4.1 Stud shearing failure

The mode of stud shearing failure is essentially the same as that for solid slab specimens described in Chapter 4.4.1. Stud shearing failure is experienced when the longitudinal shear stresses exceed the shear capacity of the stud which results in the stud shearing off in the shank adjacent to the weld. A loud noise was typically heard when a stud failed. This was accompanied by an instant sharp drop in load and successive shearing failures of the remaining studs.

However, there are also some behavioural differences which become apparent when inspecting the underside of the concrete slab after failure. In a solid slab application, the concrete surface remained sound with the exception of some localized crushing of the concrete right in front of the shear connector (see Figure 4-30). For a primary beam application, substantial longitudinal splitting and diagonal shear cracks had developed across the concrete haunch surface (Figure 5-14). In an unreinforced concrete haunch, these cracks might have led to premature failures such as longitudinal splitting failure, but as haunch reinforcement crossing the cracks was provided in the specimen shown in Figure 5-14, continuous transfer of the shear connection forces was ensured until shearing failure of the stud shank occurred. The test results presented above show that the longitudinal slips at which stud shearing occurred were usually larger in the haunch specimens (HS01-P01 to P03 in Figure 5-11, DEC1-HM04 in Figure 5-12) than in the solid slab specimen (DEC1-HM09 in Figure 5-12) which can be attributed to the reduced stiffness provided by the cracked concrete in the direct vicinity of the shear connectors. As the increased slip in the shear connectors also increases the bending effects in the shank of the stud, and as cracks weakened the concrete bearing zone, it could be concluded that the stud strength would be reduced in a haunch application as suggested elsewhere (Oehlers and Bradford 1995; Oehlers and Park 1992). However, this reduction in shear connection strength could not be observed in the test results presented above which generally experienced shear connection strengths similar to solid slab applications.
Hence, if sufficient transverse reinforcement is provided to control the propagation of the concrete cracks, the shear connection can be utilized to its full capacity.

![Splitting cracks and Diagonal shear cracks](image)

**Figure 5-14:** Underside of concrete haunch after stud shearing failure  
(Specimen HS01-P1)

Similar to secondary beam applications, stud shearing failure is also the preferred mode of failure for primary beam connections as it is the failure mode which provides far more ductility and strength than the concrete related failure modes described in the following. No increase in shear connection strength above stud shearing is possible as the stud connector material strength is fully utilized.

### 5.4.2 Longitudinal splitting failure

The load transfer of the concentrated shear connector forces into the concrete slab creates transverse tensile stresses in the concrete bearing zone which, if these stresses exceed the tensile resistance of the concrete, lead to a longitudinal splitting crack (Figure 5-15). Compared to solid slab applications, concrete haunches are of limited width and provide reduced resistance against this type of cracking. If no reinforcement is provided in the vicinity of the shear connection to transfer these transverse tensile forces after cracking has occurred, the splitting crack is free to propagate longitudinally. In the tests, it was observed that the splitting crack generally first appeared at the base of the haunch at the loaded end of the concrete
slab from where it propagated over its entire height and longitudinally along its centreline (Figure 5-16).

As all specimens were provided with transverse reinforcement, which was typically placed on top of the steel decking for the specimens with an unreinforced concrete haunch, the concrete slab did not fail completely. However, the high position of this reinforcement in the concrete slab relative to the origin of the splitting crack at the base of the shear connector meant that, at this stage, a global longitudinal splitting crack of substantial width between the bases of the individual connectors had formed resulting in a very limited stiffness against longitudinal movements in this region, i.e. the shear connection had effectively failed. Figure 5-17 shows the longitudinal splitting failure surface and stud deformations after the transverse reinforcement bars were cut. It can be seen that, distinct to the localized reverse bending effects in studs of solid slab shear connectors, tilting of the entire stud had occurred indicating the loss in longitudinal shear connection stiffness due to the position of the studs in the splitting crack. The tilting of the studs has the further effect of inducing significant amounts of tensile forces into the shanks of the studs which might result in the studs pulling out from the concrete cover slab. The separation which had occurred between the head of the shear connectors and the concrete covering the studs in Figure 5-17 is a clear indication of this secondary effect. It might also explain why many of the tested specimens experienced a combined longitudinal splitting and haunch shearing
failure (Figure 5-18) where the latter is thought to be induced by this pulling out effect (see Chapter 5.4.3).

![Image](image1.png)

**Figure 5-17:** Longitudinal splitting failure surface and stud deformations after testing (Specimen P01-P1 after transverse reinforcement bars were cut)

![Image](image2.png)

**Figure 5-18:** Combined longitudinal splitting and haunch shearing failure (Specimen DEC1-HM10)

### 5.4.3 Haunch shearing failure

The specimens experiencing haunch shearing failure all behaved in a similar fashion. A concrete crack across the full width of the haunch between the adjacent top corners
of the steel decking occurred at the loaded end of the concrete slab which resulted in a constant and continuous drop in load during the course of the further testing. After the tests were finished, the cover slab of the concrete could be easily removed from the concrete haunch and steel decking. A cone shaped failure surface had formed proceeding from the underside of the headed stud shear connectors to the upper outer edges of the concrete haunch in the transverse direction and overlapping with the failure cone of the adjacent studs in the longitudinal direction while locally undermining the bottom reinforcement mesh (Figure 5-18 and Figure 5-19). It can be seen that, for the configurations tested, a large proportion of the failure surfaces formed at very low angles to the horizontal which made the horizontal reinforcement ineffective.

The failure mode is, in many ways, similar to the stud pull-out failure observed for secondary composite beam shear connections. The longitudinal shear forces induced into the concrete slab at the bottom of the shear connector need to be transferred to the compressive zone in the top of the concrete slab. This offset causes bending effects which result in vertical tensile forces in the shear connection. The vicinity of free concrete edges and adjacent stud connectors, however, lead to a reduced effective embedment depth of the studs and enables the development of a near-
horizontal failure surface between the concrete haunch and the cover slab which cannot be reinforced by using conventional horizontal reinforcing mesh. Similar to stud pull-out failures, shear connections experiencing haunch shearing failures can behave in a very brittle manner.

5.5 Analysis of test results

5.5.1 General

Longitudinal splitting and haunch shearing failures both show similar characteristics to some of the longitudinal shear failures described in AS2327.1 (Standards Australia 2003a) with longitudinal splitting exhibiting a vertical failure surface similar to a Type 1 shear surface, and haunch shearing exhibiting a Type 3 like shear surface emanating from the top corners of the steel decking and enclosing the shear connectors. However, in contrast to a Type 3 failure surface, haunch shearing was also found to form a near horizontal shear surface in areas other than in the immediate vicinity of the stud connection. The longitudinal shear capacity of a composite beam cross-section is defined by the following parameters: concrete strength, shear surface length and, if existent, area and strength of the longitudinal shear reinforcement crossing the particular failure surface. The longitudinal shear forces experienced per unit length depend on the number and longitudinal spacing of the individual shear connectors provided and the distribution of the shear forces over the cross-section width for the shear surface investigated. The parameters for both the longitudinal shear resistance and longitudinal shear forces are also thought to have significant impact where the failure modes of longitudinal splitting and haunch shearing are being considered. Hence, these factors are investigated in regards to their impact on the shear connection strength and behaviour in the following. As only limited test results are available from the push-out tests described above, further results from recent push-out tests conducted elsewhere (Gnanasambandam 1995; Wu 1998; Yuan 1996) are also considered where appropriate.
5.5.2 Concrete tensile strength

The influence of the concrete tensile strength (given as a function of the concrete compressive strength in accordance with Equation (4-1)) on the maximum shear connection strength of unreinforced concrete haunches is shown in Figure 5-20 for various steel decking geometries. As expected, the capacity of shear connections experiencing haunch shearing and longitudinal splitting failures seemed to be strongly influenced by the concrete tensile strength which itself is a parameter subject to large variations. However, assuming a linear correlation between shear connection capacity and concrete tensile strength seems to provide a reasonable estimate for both these failure modes.

![Figure 5-20: Influence of concrete tensile strength on shear connection strength of unreinforced haunch applications for various decking geometries](image)

5.5.3 Width of concrete haunch

The width of the concrete haunch defines the failure surface for haunch shearing failure and, at the same time, reduces the longitudinal splitting resistance (see Chapter 2.3.3.2). The influence that the concrete haunch width has on the shear connection maximum strength is shown in Figure 5-21 for specimens of similar concrete strength. It can be seen that increasing the haunch width increases the
strength of shear connections subject to either haunch shearing or longitudinal splitting failures. Increasing the haunch width may also alter the failure mode experienced from haunch shearing failure for narrow haunches to longitudinal splitting failure for wider concrete haunches. A further increase in concrete haunch width above the values shown in Figure 5-21 could possibly result in a further change of failure mode from longitudinal splitting to stud shear failure such that a shear connection behaviour similar to a solid slab application might then be achieved.

![Figure 5-21: Influence of concrete haunch width on shear connection strength of unreinforced haunch applications for various decking geometries](image)

**5.5.4 Stud embedment depth**

For haunch shearing failures, which were found to propagate from the head of the stud connectors, it was thought that an increase in embedment depth of the studs into the cover slab of the concrete slab might have a beneficial effect on the shear connection strength as it increases the failure surface area. However, for the tests conducted, an increase in stud height did not result in an increase in the maximum shear connection strength nor did it alter the behaviour (Figure 5-22). However, as only one higher stud specimen was tested, further tests are required in order to verify this finding.
The longitudinal stud spacing which defines the longitudinal shear forces in a cross-section of a composite beam was not a parameter in the push-out tests described in Chapter 5.2. However, previous research investigated this parameter in detail for primary beam applications (Gnanasambandam 1995; Wu 1998). Some of the test results of these investigations are shown in Figure 5-23. It can be seen that for the various applications, the strength increased approximately linear with increasing stud spacings as indicated by the dotted linear trendlines. The failure modes observed in these tests were haunch shearing, longitudinal splitting or stud shearing. It should be noted that for the most widely spaced connectors which typically experienced stud shearing failures, the strength was still lower than the corresponding solid slab strength. As the haunches investigated were generally unreinforced, local splitting cracks in the vicinity of the shear connections, even though not resulting in a global failure, must have weakened the bearing capacity of the concrete surrounding the base of the studs and, in consequence, reduced the shear connection strength.

**Figure 5-22:** Influence of stud embedment depth on shear connection strength of unreinforced haunch applications

### 5.5.5 Longitudinal stud spacing

The longitudinal stud spacing which defines the longitudinal shear forces in a cross-section of a composite beam was not a parameter in the push-out tests described in Chapter 5.2. However, previous research investigated this parameter in detail for primary beam applications (Gnanasambandam 1995; Wu 1998). Some of the test results of these investigations are shown in Figure 5-23. It can be seen that for the various applications, the strength increased approximately linear with increasing stud spacings as indicated by the dotted linear trendlines. The failure modes observed in these tests were haunch shearing, longitudinal splitting or stud shearing. It should be noted that for the most widely spaced connectors which typically experienced stud shearing failures, the strength was still lower than the corresponding solid slab strength. As the haunches investigated were generally unreinforced, local splitting cracks in the vicinity of the shear connections, even though not resulting in a global failure, must have weakened the bearing capacity of the concrete surrounding the base of the studs and, in consequence, reduced the shear connection strength.
5.5.6 Straight ladder haunch reinforcement

The application of straight ladder reinforcement at the base of the concrete haunch was able to suppress the effects of longitudinal splitting failure and shear connection strength and behaviour similar to a solid slab application might be experienced if no subsequent concrete related failures occur (Specimen DEC1-HM04). However, by suppressing longitudinal splitting failure and increasing the shear connection strength, the haunch becomes more prone to horizontal haunch shearing failures (Specimen P02-P1). In specimens with unsplit narrow haunches where haunch shearing failures were generally experienced, the sole use of ladder reinforcement did not have any major effects on the shear connection behaviour (Specimen P03-PR1).

The results of series OBD-HS01 indicate that longitudinal reinforcement bars, as components of the bottom haunch reinforcement, would not necessarily be required as the specimen where these bars were omitted (HS01-P03) was also able to reach a shear connection strength similar to solid slab applications and experienced a stud shearing failure. However, the specimens in the OBD-HS01 series had a high concrete compressive strength ($f'_c \approx 50\text{MPa}$) and it is not sure if these findings might
also be applicable for shear connections with lower concrete strengths. Furthermore, in these specimens, the transverse ladder reinforcement was an integral part of the handle-bar component. This component might have provided additional anchorage which does not exist in sole ladder reinforcement applications.

![Graph showing load-slip behaviour and strain gauge readings](image)

**Figure 5-24:** Straight ladder haunch reinforcement strain gauge readings compared to load-slip behaviour for specimen DEC1-HM06

In Figure 5-24, the strain gauge readings for the straight ladder haunch reinforcement of one of the tested specimen (DEC1-HM06) are presented. These demonstrate the activation of all components of the haunch reinforcement. At about the end of the linear-elastic behaviour of the specimen, when the first internal cracks would have started to form, strains developed in the transverse bar in the concrete haunch. The onset of strains in the two longitudinal reinforcement bars occurred at a later stage at around 1mm slip when the shear connection had started to experience substantial plastic deformations. Thereafter, an increase in slip of the shear connection was closely related to increases in strains of all reinforcement components. However, the strains experienced in the transverse bars were generally larger than those in the longitudinal bars. At the end of the test, the transverse reinforcement bar had experienced strains well exceeding its yield strain, i.e. the bar was fully utilized, whereas the longitudinal bars were approaching their yield capacity. Hence, a smaller longitudinal reinforcement bar diameter might well be chosen for future applications.
In contrast to the results of the OBD-HS01 series, the strain gauge readings indicate that the longitudinal bars play an important part in both the anchorage of the transverse bars and the transfer of longitudinal tensile forces which occur in the concrete haunch. Further investigations regarding the use of longitudinal reinforcement bars would therefore be desirable. It should be noted that, from a constructional point of view, longitudinal bars as part of the straight haunch reinforcement component would be required regardless in order to ensure practical installation.

As most of the specimens investigated as part of the LST-DEC1 series experienced failures of the concrete cover slab while the shear connections still remained intact, no concluding statements can unfortunately be made about the influence that the placement of the ladder reinforcement or its diameter had on the shear connection behaviour. However, the strain gauge readings of the ladder reinforcement in specimen DEC1-HM05, which consisted of diameter 9.5mm transverse bars and diameter 7.6mm longitudinal bars as opposed to diameter 12mm transverse bars and diameter 9.5mm longitudinal bars in specimen DEC1-HM06 displayed similar strains as those given in Figure 5-24. As both specimens also experienced similar shear connection strengths and behaviours, the smaller ladder reinforcement component of specimen DEC1-HM05 seems to be sufficient for this particular application.

**5.5.7 Vertical haunch reinforcement**

The application of vertical haunch reinforcement generally suppressed the effects of haunch shearing failures and, used in combination with the straight haunch reinforcement, ensured very ductile shear connection behaviour. Where no concrete cover slab failures occurred, which are assumed to be unrelated to the shear connection behaviour, specimens including vertical haunch reinforcement in combination with straight ladder reinforcement generally experienced stud shearing failures and displayed behaviours similar to solid slab applications, i.e. the stud connectors were fully utilized. If sufficient vertical reinforcement was provided, this
behaviour was observed regardless of the haunch width, stud spacing or concrete strengths.

Figure 5-25 shows a longitudinal cut through a specimen which included the vertical reinforcement component (P02-P2). Diagonal cracks, which are an indication of the diagonal concrete struts which ensured load transfer from the base of the shear connection into the concrete compressive zone, are visible. In a transverse cut through the haunch of another specimen (RIB5-SDM7), the near-horizontal haunch shearing crack which had formed between the upper edges of the concrete haunch can be clearly seen (Figure 5-26). The use of vertical reinforcement (indicated by the dotted line in Figure 5-26) contained these cracks and ensured continuous load transfer by preventing a separation of the concrete haunch along longitudinal failure surfaces.

Figure 5-25: Longitudinal cut through the haunch of a specimen including vertical haunch reinforcement (P02-P2)

No conclusions can be drawn from the tests described in Chapter 5.2.2 regarding the height and diameter of this vertical haunch reinforcement as the specimens of the LST-DEC1 series that included handle-bar components generally failed due to a concrete cover slab failure. However, it is considered to be important to provide sufficient anchorage of the reinforcement component in the concrete cover slab. The component should also intersect with the top face reinforcement so as not to provide
any potential longitudinal shear failure surfaces. However, further investigations into the behaviour of vertical haunch reinforcement would be desirable.

Figure 5-26: Transverse cut through the haunch of a specimen including handle-bar haunch reinforcement (RIB5-SDM7)

5.6 Implementation of findings into shear connection design

In accordance with Section 9 of AS2327.1 (Standards Australia 2003a), longitudinal shear failures are not acceptable in composite beam design. The application of reduction factors as currently provided in most of the overseas standards (see Chapter 2.3.3.1) or the use of other available strength prediction methods as presented in Chapter 2.3.3.2 which account for the reduced strengths of primary beam shear connections experiencing longitudinal splitting or haunch shearing failures which, in many ways, were found to have similar characteristics as the longitudinal shear failures, would not be consistent with AS2327.1 (Standards Australia 2003a). Neither would the use of any of these methods be desirable as they generally do not account for the brittle effects that these concrete-related failure modes have. The aim should rather be to suppress the effects of longitudinal splitting and haunch shearing failures by providing sufficient and appropriate transverse reinforcement in order to ensure shear connection capacity and ductility consistent with solid slab behaviour.
The longitudinal shear capacity per unit length $V_L$ of a longitudinal shear surface in accordance with Section 9 of AS2327.1 (Standards Australia 2003a) is given as

$$V_L = u \left(0.36\sqrt{f_c'}\right) + 0.9A_{sv}f_{yr} \tag{5-1}$$

where $u$ is the shear surface perimeter length, $A_{sv}$ the cross-sectional area of longitudinal shear reinforcement crossing the failure surface and $f_{yr}$ the yield strength of the reinforcement. In order to extend the longitudinal shear design provision given by Equation (5-1) to haunch shearing failures, a horizontal failure surface extending between the top corners of the concrete haunch is proposed to be the shear surface with the perimeter length $u_{hs}$ as shown in Figure 5-27. From Equation (5-1), the cross-sectional area of the vertical haunch reinforcement $A_{sh,v}$ required per unit length to ensure longitudinal shear transfer can consequently determined to be

$$A_{sh,v} = \frac{1}{0.9f_{yr}} \left[ V_L - u_{hs} \left(0.36\sqrt{f_c'}\right) \right]. \tag{5-2}$$

\[\textbf{Figure 5-27:} \text{ Assumed longitudinal haunch shearing failure surface}\]

In Figure 5-28, the vertical haunch reinforcement required in accordance with Equation (5-2) is displayed as a function of the longitudinal stud spacing $s_c$ for 19mm diameter stud connections in 32MPa concrete using 240mm wide concrete haunches formed by W-Dek® steel decking. The dotted line assumes a nominal shear connector strength of $f_{vs,n}=93$ kN as given by Table 8.1 of AS2327.1 (Standards Australia 2003a) and the solid line a predicted mean solid slab shear connection strength of $f_{vs,m}=120$ kN. Even though the vertical haunch reinforcement provided in the tests was generally designed for the nominal shear connection strength, no haunch shearing failures occurred in the tested specimens. For single stud applications,
significantly higher connection forces had occurred as typically the full solid slab strength was achieved in the push-out specimens except for the specimens experiencing unrelated cover slab failures. Hence, for the limited number of tests performed, the application of Equation (5-2) provided a safe estimate of the required vertical shear reinforcement for single stud applications. The tests for stud pair shear connections are not as conclusive as cover slab failures occurred for all specimens tested at loads slightly below the nominal shear connector strength (see Table 5-7). However, the test results show that at least 70% of the solid slab strength can be achieved in stud pair applications if the required vertical haunch reinforcement is provided. It is considered that a test arrangement that prevents cover slab failures would also allow the solid slab shear connection strength to be achieved.

Figure 5-28: Vertical haunch shear reinforcement $A_{sh,v}$ for 19mm diameter studs in concrete haunches incorporating W-Dek® decking in accordance with Equation (5-2)

It is further proposed that, in contrast to the Type 2 and 3 longitudinal shear reinforcement provisions of AS2327.1 (Standards Australia 2003a), no minimum longitudinal shear reinforcement needs to be provided against haunch shearing failure in single stud applications if the occurring longitudinal shear forces can be transferred by the concrete strength alone, i.e. the bracketed term in Equation (5-2) is determined to be negative. For the specimen where no vertical haunch reinforcement was theoretically required, as it had a wider stud spacing, and which subsequently
did not contain any such reinforcement (DEC-HM04), solid slab shear connection strength was still experienced with no signs of haunch shearing failure. Note that the minimum shear reinforcement requirements are also waived in other applications such as the shear design of concrete slabs in accordance with AS3600 (Standards Australia 2001).

The application of Equation (5-1) for the design of the transverse haunch reinforcement to prevent longitudinal splitting failure considering a vertical failure surface in the concrete haunch similar to a Type 1 failure surface in accordance with AS2327.1 (Standards Australia 2001) can currently not be recommended as the specimens tested to date all contained more transverse haunch reinforcement than Equation (5-1) would have required, particularly for single studs. The transverse splitting reinforcement provided in the LST-DEC1 specimens are compared with the requirements of Equation (5-1) in Figure 5-29, assuming that 50% of the required Type 1 reinforcement would be placed in the concrete haunch. Further tests would be required to verify an eventual application of Equation (5-1) to reinforce against longitudinal splitting. However, as the mechanism of the longitudinal shear transfer in reinforced concrete on which Equation (5-1) is based and the mechanism of a concentrated load transfer into a narrow concrete section which causes longitudinal splitting failure are substantially different, agreement might not be achieved.

A method to account for the reinforcement provided in longitudinally split concrete slabs derived by Oehlers and Park (1992) was given in Equations (2-4) to (2-7) in which an upper limit for the shear connection strength in cracked slabs was set to $f_{vs,cr} \leq 0.9 f_{vs}$. As, in the tests presented above, the shear connection in slabs incorporating transverse haunch reinforcement experienced strengths similar to solid slab applications, this upper limit is not further considered. In order to obtain strengths of $f_{vs,cr} = f_{vs} = P_{solid}$, the following transverse haunch reinforcement per unit length $A_{sh,h}$ would be required in accordance with Equations (2-4) and (2-5):

$$A_{sh,h} \geq 0.13 \frac{h_s^2}{s_c} = 0.43 \frac{d_{sh}^2}{s_c}. \quad (5-3)$$

However, in the tests performed, substantially less haunch reinforcement than required by Equation (5-3) was provided without the specimens experiencing reduced shear connection strength due to longitudinal splitting (see Figure 5-29).
Equation (2-4) was based on the premise that the shear connection strength after longitudinal splitting occurs depends on the stiffness of the transverse reinforcement and not upon its strength. Hence, parameters which account for the strength of the splitting reinforcement and the applied longitudinal shear forces are absent in this approach. The measurements of high strains in the haunch reinforcement bars which indicated yielding of the transverse bars (see Chapter 5.5.6) contradicts this presumption of pure stiffness requirements for the design of transverse haunch reinforcement.

In the latest version of the German composite standard DIN V 18800-5 (DIN 2004), a simple force equilibrium requirement for the transfer of the existing tensile splitting forces in haunches with split decking is given for primary beam applications. Therein, the transverse haunch reinforcement is required to transfer at least 30% of the existing stud forces:

$$A_{sh,h} f_{yr} \geq 0.30 \frac{n_s f_{ys}}{s_c}.$$  \hspace{1cm} (5-4)

A similar requirement is also used for the transverse reinforcement design of horizontal lying studs in thin concrete slabs (see also Breuninger and Kuhlmann (2001)). In accordance with DIN V 18800-5 (DIN 2004) the maximum allowable longitudinal spacing of the transverse bars for lying studs is $18d$. In the absence of
available test data, it is proposed to also apply this limitation to the design of transverse haunch reinforcement. It is interesting to note that, in a related application, AS3600 (Standards Australia 2001) requires only up to 25% (rather than 30%) of longitudinal anchorage forces in post-tensioned concrete members to be transferred as a transverse tensile force.

The provisions of DIN V 18800-5 (DIN 2004) seem to best represent the haunch reinforcement provided in the tests, particularly for single studs (Figure 5-29). The reinforcement provided was generally found to have successfully suppressed the effects of longitudinal splitting failure. Until more test data becomes available, it is therefore recommended to design the bottom transverse haunch reinforcement in accordance with Equation (5-4). In addition, the reinforcement should not be placed higher in the concrete haunch than at its mid-height. Note that in order to prevent global splitting failure of the concrete slab, the vertical Type 1 shear failure requirements of AS2327.1 (Standards Australia 2003a) still need to be considered, irrespective, for the transverse reinforcement design.

### 5.7 Summary

The haunches created in primary beam applications of Australian types of trapezoidal steel decking might cause premature concrete-related failure modes such as longitudinal splitting and haunch shearing. These types of failures can lead to reduced strength and a very brittle response of the shear connection. The application of conventional longitudinal shear reinforcement placed on top of the steel decking proved to be ineffective in containing the effects of these failures. Where a horizontal type of ladder reinforcement was provided in the concrete haunch, the effects of longitudinal splitting failures were successfully suppressed and a shear connector strength and behaviour similar to solid slab applications was experienced if no successive haunch shearing failures occurred. Haunch-shearing failures, which are characterized by near-horizontal failure surfaces between concrete haunch and cover slab, were successfully contained by providing an additional vertical stirrup or handle-bar reinforcement similar to haunch applications in solid slabs. Again, the full
shear connector strength could be utilized and a ductile behaviour was ensured unless concrete slab failures, unrelated to the shear connection behaviour, occurred.

The analysis of the test results revealed that the strength of shear connections in unreinforced concrete haunches is strongly affected by the concrete tensile strength, haunch width and longitudinal stud spacing whereas the embedment depth of the stud into the concrete cover slab did not seem to have a strong influence on the connection behaviour. Hence, the parameters influencing the premature primary beam failures are similar to those considered in the longitudinal shear design of composite beams. Based on the limited number of test results available, preliminary design suggestions for the implementation of the findings into Section 9 of AS2327.1 (Standards Australia 2003a) are made with the aim to adequately reinforce the concrete haunches, incorporating the Australian types of trapezoidal steel decking, against longitudinal splitting and haunch shearing failures.

Further tests are strongly recommended to closer study the different parameters such as haunch dimensions, shear connector arrangement, sheeting thickness and reinforcement dimensions and positions and to verify the reinforcement requirements for haunches in composite beam applications.
6 DESIGN MODELS FOR SECONDARY BEAM STUD CONNECTIONS

6.1 Introduction

It is the aim to establish a reliable method to determine the capacity of headed stud shear connections which differentiates between the ductile and brittle failure modes for the trapezoidal types of Australian steel decking available. Whenever a brittle failure mode is identified as the governing failure mode, the shear connection should generally not be acceptable for plastic design unless a minimum strength can be guaranteed at the required slip capacity. Otherwise, suitable reinforcing measures need to be applied in order to suppress the effects of this failure mode and increase its ductility to a satisfactory level.

Numerous methods to determine the strength of shear connectors in secondary beam applications already exist. Some were presented in Chapter 2.3.2 and most were derived in a semi-empirical manner by evaluating the maximum strength of push-out test results. However, in contrast to previous steel deckings, the new types of Australian decking display significant differences such as the use of a high strength G550 galvanized steel sheeting or, in case of the KF70 decking, a small re-entrant rib on top of the large trapezoidal steel rib. Therefore, the applications of some of the approaches used overseas are re-evaluated in regards to the new types of steel decking and to the application of the shear connection reinforcing elements. Also, some recent approaches, which distinguish between the various failure modes experienced in shear connections and which are partly based on individual
Design models for secondary beam stud connections

mechanical models, are evaluated for their possible application. The application of these approaches should make it easier to differentiate between ductile and brittle shear connection behaviour.

However, as no satisfactory correlation between the test results presented in Chapter 4 and any of the methods evaluated could be found, a novel method to determine the shear connection capacity is developed for secondary beam applications which takes into account the various factors influencing the behaviour, as described in Chapter 4.5.2, and the beneficial effects of the application of suitable reinforcing elements. The key element of the approach is to classify the anticipated connection behaviour, in regards to its deformation capacity, into ductile or brittle, hence ensuring satisfactory shear connection behaviour where trapezoidal types of steel decking are used.

6.2 Evaluation of existing models

6.2.1 Reduction factor approaches

6.2.1.1 General / Solid slab strength

Most of the design approaches currently used around the world take into account the weakening effect of trapezoidal types of decking by applying a reduction factor to the nominal strength that the same connection would have in a solid slab. Consequently, it is critical that the strength and behaviour of the connector in the solid slab is known and understood. In Ernst et al. (2005), an extensive review of previous investigations into the shear connection behaviour in solid slabs was undertaken in order to determine the mean strength of headed stud shear connectors in solid slabs. The mean stud strengths of these previous investigations and of the latest test data available were calibrated to the reduction factor approach given by Equations (2-1) and (2-2). The findings for the mean stud strength calibration factors $c_{1m}$ and $c_{2m}$ in the various investigations are summarized in Table 6-1.
Table 6-1: Summary of the mean stud strength factors determined by different researchers (Ernst et al. 2005)

<table>
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</thead>
<tbody>
<tr>
<td>$c_{1m}$</td>
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<td>N/A</td>
<td>0.44</td>
<td>0.48</td>
<td>0.44</td>
<td>0.47</td>
</tr>
<tr>
<td>$c_{2m}$</td>
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<td>0.92-0.97</td>
<td>0.91</td>
<td>1.00</td>
<td>0.92</td>
<td>0.94</td>
</tr>
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As a simple guideline, the average value of these investigations is assumed to provide a reasonably accurate measure for the determination of the mean stud strength leading to the following provisions for solid slab shear connections:

\[
f_{vs,m} = 0.47 \sqrt{f_{c} E_{c} A_{sc}}
\]  \hspace{1cm} (6-1)

\[
f_{vs,m} \leq 0.94 f_{sc} A_{sc}
\]  \hspace{1cm} (6-2)

The maximum difference to the values determined in the various investigations would then amount to approximately ± 6%.

The calibration factors given in Table 6-1 highlight that the stud strengths provided by AS 2327.1 (Standards Australia 2003a) or other standards do not reflect the mean shear connection strength. Even if the measured material strengths rather than their characteristic values are used in Equations (2-1) and (2-2), the mean stud strengths determined in accordance with Table 6-1 would still be between 18% and 21% higher than the strength determined in accordance with the design provisions of AS2327.1 (Standards Australia 2003a) (compare also Table 2-1). Note that the overall reliability of the solid slab design provisions, as given in AS2327.1 (Standards Australia 2003a), is evaluated in Chapter 8. For the following assessment of the reduction factor approaches, the mean solid slab shear connection strength given by Equations (6-1) and (6-2) is used.

6.2.1.2 Eurocode 4 method

The experimental strength of the secondary beam shear connections described in Chapter 4.2 incorporating the novel Australian types of trapezoidal steel decking are compared in Figure 6-1 and Figure 6-2 with the predicted strength $f_{vs,EC4}$ calculated
Design models for secondary beam stud connections

using the reduction factor approach according to Eurocode 4 (CEN 2004) which is described in detail in Chapter 2.3.2.2. Figure 6-1 also includes the specimens where the steel decking was omitted as the application of the reduction factor does not require a minimum sheeting thickness. In Figure 6-2, some of the results of push-out tests performed by Oehlers and Lucas (2001) on the KF70 steel decking type were also included. The 15 mm high, narrow dovetail rib on top of the main steel rib in this decking was generally ignored as it does not alter the geometry of the main trapezoid concrete rib into which the shear connection is placed and it is believed to not have a direct influence on the shear connection behaviour. However, the overall height of the profiled steel decking has a profound impact on the predicted shear connection strength in the Eurocode 4 (DIN 2004) approach. The shear connection strength is significantly reduced with an increasing rib height. Hence, several researchers have recently recommended the use of the total height of the steel decking when using this approach (Bridge and Patrick 2002; Johnson 2005; Kuhlmann and Raichle 2006), solely owing to the reduced shear connection strength but for no apparent technical reasons.

![Figure 6-1: Comparison of shear connection strength incorporating W-Dek® concrete rib geometry with Eurocode 4 (CEN 2004) method](image_url)
For both types of steel decking geometry, the application of the Eurocode 4 (CEN 2004) reduction factor to the mean strength that the same connection would have in a solid slab does not provide a very accurate prediction. For conventionally reinforced shear connections, it overestimates the strength and allows for shear connections which do not fulfill the ductility requirements of $\delta_u$ exceeding 6 mm as given in Appendix B of Eurocode 4 (CEN 2004). The non-ductile specimens are marked with a black cross. Where the stud enhancing device was applied, the results are less distinctive as several specimens reached the predicted strength but many still displayed insufficient ductility. The use of the waveform element provided satisfactory ductility and a conservative strength estimate for all but one specimen. In this narrow slab W-Dek® specimen, a rib shearing failure was observed in one of the ribs, despite the presence of the waveform element, which might indicate insufficient anchorage of the component. The combined application of the stud enhancing device and the waveform element resulted in sufficient ductility and a conservative strength prediction for all specimens incorporating W-Dek® geometry regardless of internal or edge beam applications. For the specimens incorporating the KF70® geometry, two stud pair specimens did not fulfill the ductility criteria. They experienced a rib shearing failure where the failure surface propagated between the waveform element and top reinforcement layer indicating insufficient anchorage of the waveform element.
reinforcement. In regards to the two decking geometries tested, it can be concluded that the provisions of Eurocode 4 (CEN 2004) should only be applied for shear connections where the waveform element is used, be it on its own or in combination with a stud enhancing device. However, it should be ensured that the component is properly anchored in the concrete cover slab.

The safety concerns of the Eurocode 4 reduction factor method, when used for conventional reinforced secondary beams shear connections, are not restricted to Australian types of steel decking. The experimental strength of 119 conventional reinforced push-out tests with the steel decking running transverse to the beams (Hanswille 1993; Rambo-Roddenberry 2002; Roik et al. 1988; Yuan 1996) are compared in Figure 6-3 with the predicted stud strength according to Eurocode 4 (CEN 2004). Again, the reduction factor was applied to the mean strength that the same connection would have in a solid slab. Only test specimens with a steel decking height $h_r$ of 80mm or less and with a stud diameter $d_{bs}$ ranging from 16mm to 22mm were considered. The results are alarming, not just as the majority of the test results did not reach the predicted mean strength but also for the large scatter in the results. Furthermore, using the approach outlined in Eurocode 4 (CEN 2004), 38 of the 82 specimens where deformation data was available did not fulfil the ductility requirement. The non-ductile specimens, marked with a black cross, indicate that the few specimens exceeding the predicted strength mostly behaved in a brittle manner.

It is realised that in the design method in Eurocode 4 (CEN 2004), the reduction factor is applied to the nominal design strength that the connection would have in a solid slab which is less than the mean strength. If this lower design strength was used, incorrectly, as the predicted strengths, the results in Figure 6-1 to Figure 6-3, would move to the left giving an apparently better fit to the experimental results, but it would not improve the large scatter. Hence, the limited geometric parameters used in the reduction factor are not sufficient to model the experimental behaviour, a fact also recognized elsewhere, e.g. Johnson and Yuan (1998a); Kuhlmann and Raichle (2006). It should be noted that the ratio of experimental to nominal stud strength was used in the statistical analysis to determine an appropriate safety factor in Eurocode 4 (Stark and van Hove 1991).
6.2.1.3 ANSI-AISC 360-05 method

The results of the secondary beam push-out tests presented in Chapter 4.3 and the internal beam test results published by Oehlers and Lucas (2001) are compared with the very recent American reduction factor approach of ANSI-AISC 360-05 (AISC 2005) in Figure 6-4 and Figure 6-5. The approach is presented in Chapter 2.3.2.2. It can be seen that the scatter is very large; hence many of the factors affecting the shear connection behaviour seem not to be reflected in this approach. In the push-out tests described in Chapter 4, it was observed that all premature failure modes are concrete-related and the strength was strongly affected by the concrete tensile strength. In contrast, where this method is applied, the shear connection strength is independent of the concrete properties. The reduction factor also does not account for other important parameters such as steel decking height or thickness. For most of the conventional reinforced shear connections, the shear connector strength is overestimated whereas for the shear connections incorporating the stud enhancing device, the waveform element or the combination of both, no distinctive trends can be recognized. It is generally not recommended to use this method when designing shear connections using Australian types of profiled trapezoidal steel decking as it
provides unconservative estimates for most applications, even for some of the applications where reinforcing elements were used.

![Figure 6-4: Comparison of shear connection strength incorporating W-Dek® concrete rib geometry with the ANSI-AISC 360-05 (AISC 2005) method](image1)

![Figure 6-5: Comparison of shear connection strength incorporating KF70® concrete rib geometry with the ANSI-AISC 360-05 (AISC 2005) method](image2)
6.2.2 Failure mode approaches

Models which take into account the effects and factors influencing the various failure modes experienced in the shear connection are far more desirable in comparison to the reduction factor approaches. Two recent methods (Jenisch 2000; Johnson and Yuan 1998b) which are presented in Chapter 2.3.2.3 are assessed for their applicability for shear connections incorporating the two Australian types of steel decking geometries and the various reinforcing elements investigated in Chapter 4.

6.2.2.1 Johnson and Yuan model

The approach distinguishes between three possible failure modes: rib punch-through, stud pull-out and stud shearing. Where stud pairs are placed in a diagonal position in concrete ribs, a combined rib punch-through and stud pull-out failure also needs to be investigated. For this case, the stud placed in the unfavourable position is thought to initially experience rib punch-through failure followed by the stud in the favourable position which is thought to experience stud pull-out failure.

Shear connections subject to stud shearing failure are assumed to experience the same stud strengths as solid slab connections. The models for stud pull-out and rib punch-through failures were presented in detail in Chapter 2.3.2.3 and the shear connection capacities against these failures can be determined in accordance with Equations (2-21) and (2-31) respectively. However, in both provisions, the capacity is given as a function of the tensile force $T_c$ acting in the shank of the stud. The approach assumes that only negligible tensile forces act in the shank of a solid slab shear connection. The interaction between the stud tensile force and its shear strength in a composite slab application is then based on a von Mises-type of interaction:

$$\left(\frac{f_{sy}}{f_{sy, solid}}\right)^2 + \left(\frac{T_c}{T_{yc}}\right)^2 = 1 \quad (6-3)$$

where $T_{yc}$ is the yield strength of the stud connector shank and can be approximated to
If Equation (6-3) is substituted in Equations (2-21) or (2-31), the shear connection capacity against stud pull-out and rib punch-through failures can then be directly obtained providing the solid slab shear connection strength is known.

However, the application of Equation (6-3) indicates that a reduction in horizontal shear connection strength automatically implies an increase in the tensile forces acting in the shank of the stud. For example, if a composite slab shear connection would only experience half the solid slab strength, the stud shank would theoretically be loaded to 87% of its tensile capacity. This interaction between tensile forces and shear connection strength could generally not be observed in the axial strain measurements in the stud shanks (see Chapter 4.5.2.10). It was rather found that the ratio of the stud tensile forces to the shear connection strength was more or less constant and independent of the failure mode experienced. For the strain measurements undertaken, the tensile forces in the stud shanks amounted to 7.5kN-33.5kN (see Table 4-27) which would equal 6.5% to 29% of the stud yield strength as given by Equation (6-4). The corresponding measured shear strengths for these two specimens were 52% and 49% of the expected solid slab strength as opposed to the theoretical predictions of 99.8% and 96% if Equation (6-3) is applied. These test results indicate that the form of interaction of Equation (6-3) is not suitable for connections that experience premature concrete related failures. This is probably because the interaction between stud tensile and shear capacity given in Equation (6-3) has originally been derived for stud shank failures (fracture in combined shear/tension) analogous to failure of a bolt.

Nevertheless, the approach is assessed against the push-out test results of the two rib geometries tested in Chapter 4 in Figure 6-6 and Figure 6-7 in which the value $f_{\text{vs,Johnson/Yuan}}$ reflects the minimum of the shear connection capacities against stud shearing, stud pull-out and rib punch-through failure. The mean solid slab strength was determined in accordance with Equations (6-1) and (6-2). For the KF70® geometry, the height of the main trapezoidal rib was assumed to best reflect the height of the concrete rib ($h_r=55\text{mm}$). As the studs for this type of decking geometry were generally placed alternately between a central position in the pans without a lap...
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joint and the designated direction in the other pans, the minimum distance $e$ of these two spacings was used in the determination of the rib punch-through strength. All shear connections incorporating the W-Dek® type of steel decking had the steel decking pre-holed. However, it was still decided to take the steel decking component of Equation (2-31) into account as the fracture of the steel decking in some of the specimens clearly indicated that the decking participated in the load transfer (see Chapter 4.5.2.5).

![Figure 6-6: Comparison of shear connection strength incorporating W-Dek® concrete rib geometry with Johnson and Yuan (1998b) model](image)

The assessment shows, that in particular for the KF70® decking geometry, the trend of the shear connection strength seems to be better reflected than in the previous reduction factor approaches. However, the scatter is still considerable and the approach mostly seems to overestimate the shear connection strength. It should be noted that the approach does not consider the beneficial effect of waveform reinforcement nor was it calibrated against shear connections where the steel decking was omitted. For the specimens without sheeting, the approach generally provided very low shear capacities and predicted rib punch-through failures for all specimens, regardless of the number of shear connectors per concrete rib. Hence, the approach assumes a much bigger impact of the thickness of the steel decking on the predicted shear connector strength than was experienced in the tests. In the tests, the presence
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of the W-Dek® sheeting was found to increase the shear connection strength about 20kN for the conventionally reinforced specimens (see Chapter 4.5.2.5) whereas the approach predicted increases of around 50kN-60kN. The differences cannot be explained with the pre-holing of the decking if the maximum possible tensile force transfer from the stud directly into the steel decking is estimated to amount to $T_{sh,max} \approx 0.5\pi d_{weld} f_{ysh} = 22$ kN per mm of sheeting thickness for a stud weld of 25mm diameter.

Hence, the majority of the steel decking forces would need to be transferred in an indirect manner into the decking. The use of a high tensile strength galvanized G550 steel sheeting for the Australian types of decking compared to lower strength steel decking for which this approach was calibrated may also provide an explanation for this unconservative approximation.

![Comparison of shear connection strength incorporating KF70® concrete rib geometry with Johnson and Yuan (1998b) approach](image)

**Figure 6-7:** Comparison of shear connection strength incorporating KF70® concrete rib geometry with Johnson and Yuan (1998b) approach

Overall, it can be concluded that shear connections including the two types of Australian steel decking investigated should generally not be designed using this approach in its present form as it mostly underestimates the connection strength and neither provides a realistic modelling of the stud tensile forces and the forces transferred by the steel decking nor does it necessarily predict the correct failure mode.
6.2.2.2 Jenisch model

The models derived by Jenisch (2000) to predict stud pull-out and rib punch-through failures are presented in Chapter 2.3.2.3. They are based on the strut and tie model shown in Figure 2-10. The secondary beam test results of the two trapezoidal steel decking geometries investigated are compared with this approach in Figure 6-8 and Figure 6-9. Again, the scatter is very large, but the approach generally seems to provide very conservative estimates for the shear connection strength. It was found that, in particular, the contribution of the steel decking in the case of rib punch-through failure is grossly underestimated as it typically only amounted to an additional increase in strength between 1kN-3KN whereas in the tests it accounted for around 20kN increase (see Chapter 4.5.2.5). On the other hand, the tensile forces which were experienced in the stud shanks hand seem to be overestimated by this approach as stud tensile forces as high as 80kN were predicted compared to the actual measurements which amounted only to about 10-30kN (see Chapter 4.5.2.10). The approach does not take into account the strengthening effects that the reinforcing elements have and it also does not accurately predict the failure mode which was experienced.

Figure 6-8: Comparison of shear connection strength incorporating W-Dek® concrete rib geometry in secondary beam applications with Jenisch (2000) approach
As the approach seems to give a lower conservative estimate concerning the stud strength, it could be applied in the shear connection design of the Australian types of steel decking to determine if any premature failure modes other than stud shearing failure would occur. If this is the case, it is recommended to assume that the connection behaves in a brittle manner and reinforcing elements such as waveform reinforcement or a combination of waveform reinforcement and the stud enhancing device should be applied. The shear connection strength of the then sufficiently ductile connection could theoretically be determined using this approach, but the strength would be grossly underestimated.

**6.2.3 Summary**

The evaluations of the push-out test results presented in Chapter 4 with the various models for the determination of the shear connection capacity are summarized in Table 6-2 where \( n \) is the number of test results considered and \( P_m/P_t \) is the ratio of measured to predicted strengths (mean value). The approaches of Eurocode 4 (CEN 2004), ANSI-AISC 360-05 (AISC 2005) and the general method by Johnson and Yuan (1998b) all seem to significantly overestimate the shear connection strength in
conventionally reinforced specimen whereas the approach by Jenisch (2000) provides very conservative results. The coefficient of variation $V_\delta$ representing the scatter of the test results was found to be unacceptably large for all of the approaches. As only the specimens incorporating the waveform element or the enhancing device in combination with the waveform element consistently fulfilled the ductility requirements of a sufficient ductile shear connection (determined in accordance with Appendix B of Eurocode 4 (CEN 2004), a conservative estimate of the strength of these types of shear connections could be determined by using the design provisions of either the Eurocode 4 (CEN 2004) approach or the Jenisch (2000) model. Where an ideal-plastic shear connection behaviour is assumed as part of the composite beam design, none of the models investigated should be applied if conventionally reinforced shear connections incorporating the two trapezoidal types of Australian steel decking are used. However, the method of Jenisch (2000) might be employed to get an indication whether a brittle behaviour of the shear connection might be expected. A more accurate determination of the shear connection strength and behaviour, which additionally accounts for the various reinforcing elements, can be obtained using the proposed new model introduced in Chapter 6.3. In applications where a minimum shear connection strength can be ensured at large deformations, the new model additionally allows for the application of conventionally reinforced shear connections even if a concrete-related failure mode is experienced.

**Table 6-2:** Mean $P_m/P_t$ and coefficient of variation $V_\delta$ of various models for the determination of secondary beam shear connection capacity

<table>
<thead>
<tr>
<th>Model</th>
<th>Con conventionally reinforced specimen</th>
<th>Specimens incl. enhancing device</th>
<th>Specimens incl. waveform element</th>
<th>Specimens incl. enhancing device + waveform element</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$n$</td>
<td>$P_m$/$P_t$</td>
<td>$V_\delta$</td>
<td>$n$</td>
</tr>
<tr>
<td>Eurocode 4 (CEN 2004)</td>
<td>32</td>
<td>0.86</td>
<td>0.190</td>
<td>16</td>
</tr>
<tr>
<td>ANSI-AISC 360-05 (AISC 2005)</td>
<td>32</td>
<td>0.85</td>
<td>0.248</td>
<td>16</td>
</tr>
<tr>
<td>Johnson/ Yuan (1998b)</td>
<td>32</td>
<td>0.92</td>
<td>0.264</td>
<td>16</td>
</tr>
<tr>
<td>Jenisch (2000)</td>
<td>32</td>
<td>1.23</td>
<td>0.164</td>
<td>16</td>
</tr>
<tr>
<td>Proposed model (Chapter 6.3)</td>
<td>32</td>
<td>1.10</td>
<td>0.151</td>
<td>16</td>
</tr>
</tbody>
</table>

Highlighted cells: Applications for which the method provides conservative strength prediction and ductile shear connection behaviour
6.3 Proposed new method

6.3.1 General

In Chapter 4.5, it was shown that the various failure modes experienced in a secondary beam shear connection resulted in different shear connection behaviours and slip capacities. The key element of this new method is to classify the anticipated connection behaviour, in regards to its deformation capacity, into ductile or brittle connections (see Table 6-3). Whenever a brittle connection is identified, it is not applicable for plastic composite beam design. Generally, there are two possible measures to alter the shear connection behaviour from brittle to sufficiently ductile without changing the basic connection layout. One is to reduce the shear connection capacity to an amount which can be guaranteed at any given slip up to the required slip capacity. This measure is only applicable for specimens experiencing rib punch-through failures as the shear connection strength, which drops when the initial failure is experienced, is able to recover with increasing slips. Consequently, the minimum value after the first drop in load is experienced, $P_{RPT,\text{min}}$, can be taken as the shear connection strength (see Figure 6-10). The other measure is the application of reinforcing elements in the shear connection as given in Table 6-3.

<table>
<thead>
<tr>
<th>Failure mode</th>
<th>Stud capacity</th>
<th>Shear connection behaviour</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Conventionally reinforced</td>
</tr>
<tr>
<td>Rip punch-through</td>
<td>$P_{RPT,\text{max}}$/$P_{RPT,\text{min}}$</td>
<td>brittle / ductile</td>
</tr>
<tr>
<td>Rib shearing</td>
<td>$P_{RS}$</td>
<td>brittle</td>
</tr>
<tr>
<td>Stud pull-out</td>
<td>$P_{SP}$</td>
<td>brittle</td>
</tr>
<tr>
<td>Stud shearing</td>
<td>$P_{\text{solid}}$</td>
<td>ductile</td>
</tr>
</tbody>
</table>

1. $P_{RPT,\text{min}}$: Capacity which can be guaranteed at any given slip up to the required slip capacity
Design models for secondary beam stud connections

Figure 6-10: Definition of $P_{RPT,max}$ and $P_{RPT,min}$ for shear connections experiencing rib punch-through behaviour

The structure of the method is the following:

- Determination of the shear connection capacities for the various failure modes using the appropriate model ($P_{RPT,max}$, $P_{RS}$, $P_{SP}$, $P_{solid}$).
- The minimum of these capacities provides the expected failure mode.
- From Table 6-3, the expected shear connection behaviour can be obtained.
- If the expected shear connection behaviour is ductile, the shear connection is suitable for application and the predicted shear connector strength is given as $f_{vs,m}=\min(P_{RPT,max}, P_{RS}, P_{SP}, P_{solid})$.
- In the case of an expected rib punch-through failure and an initial brittle response, a ductile behaviour can be modelled by reducing the predicted shear connector strength to $f_{vs,m}=P_{RPT,min}$.
- In all other cases where the expected shear connection behaviour is brittle, the connection requires redesign either by changing its layout or by applying suitable reinforcing measures.

For the determination of the shear connection strength against stud shearing, the expected mean strength $P_{solid}$ can be obtained from Equations (6-1) and (6-2) using mean material properties. The expected mean capacities of the other failure modes are derived in the following. The appropriate design resistance factors for the various shear connection capacities which enable the application of the new method in accordance with AS2327.1 (Standards Australia 2003a) are determined in Chapter 8.
6.3.2 Rib punch-through capacity

6.3.2.1 Analogy to fastener application close to a free edge

The behaviour of rib punch-through failures has distinct similarities with the application of anchor bolts located close to a free concrete edge and subject to lateral shear forces (Figure 6-11). In both applications, diagonal cracks propagate from the fastener to the free concrete edge which eventually results in the breakout of a concrete wedge in front of the fastener. In fastener applications, it was found that the angle \( \alpha \) of this crack is subject to large variations but usually amounts to an angle \( \alpha \) between 20° to 40° for studs relatively close to the free concrete edge (Fuchs 1992). With increasing edge distance \( e \) of the shear connector, the angle was also found to increase. In accordance with Zhao (1995), the interaction between edge distance and failure wedge angle can be expressed as

\[
\tan(\alpha) = 0.2e^{0.25}.
\]  

(6-5)

Figure 6-11: Analogy of secondary beam shear connection to fastener application close to a free concrete edge

For some of the tested specimens, the angles of the failed concrete wedges were also measured after failure and are summarized in Table 6-4. For the application of Equation (6-5), the edge distance was taken as the distance from the studs to the mid-height of the steel decking \( e \). For both geometries, the failure wedge seemed to be slightly wider (smaller \( \alpha \)) if stud pairs were used. Equation (6-5) does not account
for these differences but otherwise seems to predict the failure surface width reasonably well.

Table 6-4: Angles of failed concrete wedges

<table>
<thead>
<tr>
<th>Application</th>
<th>No of measured wedges</th>
<th>Measured $\alpha$ $[^\circ]$</th>
<th>Average $\alpha$ $[^\circ]$</th>
<th>Predicted $\alpha$ (by Eq. (6-5)) $[^\circ]$</th>
</tr>
</thead>
<tbody>
<tr>
<td>KF70® decking, single studs</td>
<td>5</td>
<td>32-44</td>
<td>38</td>
<td>31/33(^1)</td>
</tr>
<tr>
<td>KF70® decking, stud pairs</td>
<td>4</td>
<td>30-38</td>
<td>34</td>
<td>31/33(^1)</td>
</tr>
<tr>
<td>W-Dek® decking, single studs</td>
<td>16</td>
<td>26-46</td>
<td>37</td>
<td>32</td>
</tr>
<tr>
<td>W-Dek® decking, stud pairs</td>
<td>16</td>
<td>19-40</td>
<td>30</td>
<td>32</td>
</tr>
</tbody>
</table>

\(^1\): central/favourable stud position

However, there are fundamental differences between both applications. The concrete rib in a secondary beam shear connection defines the vertical extension of the concrete wedge whereas the failure wedge in a fastener application was found to have an overall height of about 1.5 times the edge distance of the anchor (Eligehausen and Mallee 2000; Fuchs 1992). Furthermore, the solid cover part of the slab ensures an additional load transfer even after the concrete wedge has failed. This component does not exist in a fastener application.

6.3.2.2 Load in concrete wedge

Initially, the majority of the horizontal shear forces are transferred into the surrounding concrete at the base of the stud over an effective connector height $h_{ec}$. The effective height was found to be a function of the stud diameter and the concrete strength and density (see Chapter 2.2.1). In accordance with Oehlers (1989), the effective stud height for normal-weight and normal-strength concrete can be approximated as

$$h_{ec} \approx 1.8d_{ba}.$$  

(6-6)

If it is now assumed that the shear forces are evenly transferred over the effective stud height and that the bearing stresses of the shear connection are distributed into the concrete slab in a similar fashion as the stresses in a fastener application, which
means they spread vertically with a ratio of 1:1.5, the horizontal shear forces $P_{\text{tot}}$ can be split into two components: one which is transferred into the concrete rib, $P_{\text{wed}}$, and the other one which is transferred into the cover slab, $P_{\text{cover}}$ (see Figure 6-12). If only the horizontal component of the force $P_{\text{wed}}$ is considered, its proportion of the total shear force $P_{\text{tot}}$ can then be determined as

$$\frac{P_{\text{wed}}}{P_{\text{tot}}} = \frac{h_{e}}{h_{ec} + 1.5e_{t}}. \quad (6-7)$$

![Figure 6-12: Splitting of horizontal shear force in two components](image)

If rib punch-through failure occurs, the total shear force is immediately reduced by the amount of shear force acting in the concrete rib whereas the cover slab component still ensures continuous load transfer, hence

$$P_{\text{min}} = \left(1 - \frac{P_{\text{wed}}}{P_{\text{tot}}}\right)P_{\text{tot}} = P_{\text{cover}}. \quad (6-8)$$

In the further course of testing, the base of the stud is free to move longitudinally while the remaining stud is still embedded into the concrete cover slab which results in bending deformations of the shank of stud and changes in the effective stud height. This ultimately might lead to another increase in shear force at larger slips. However, the minimum load capacity which can be ensured at any given slip up to the required slip capacity, $P_{\text{min}}$, is required for the ideal-plastic shear connection design (see Figure 6-10).

For the specimens where the steel decking was omitted and where no subsequent failure modes were observed after rib punch-through failure occurred, the measured minimum strength $P_{\text{min}}$ relative to its measured maximum strength $P_{\text{max}}$ after the
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The capacity of the concrete wedge against break-out failure can be determined by integrating the tensile stress distribution at the time of failure over the predicted failure surface. The stress distribution of the tensile stresses $\sigma_t$ can be assumed to be a function of the distance $e$ from the shear connector:

$$P_{wed} = \int_{A_{wed}} \sigma_t(e) \, dA.$$

(6-9)
The predicted failure surface of the concrete wedge is shown exemplary for a stud pair connection in Figure 6-13. It can be assumed to generally propagate from the edges of the shear connector diagonally towards the free edge of the concrete rib forming an angle $\alpha$ to the transverse direction of the connection, as given by Equation (6-5). However, if the width of the concrete slab $b_{cf}$ is smaller than its effective failure wedge width $b_{ewt}$ then the crack is assumed to propagate diagonally towards the corner of the concrete rib and the concrete slab edge, hence $\alpha$ needs to be reduced accordingly.

$$b_{ewt} \leq b_{cf}$$

Where pairs of studs are used, the failure cones of the individual shear connectors overlap if the studs are spaced closely together (see also Chapter 4.5.2.4). In that case, the shear connection failure surfaces are assumed to form one combined concrete wedge as shown in Figure 6-13. In the vertical direction, the height of the wedge is generally limited to the height of the concrete rib, $h_r$, as observed in tests. The sides of the wedge are assumed to be vertical. This creates three failure surfaces, one at the top ($A_{top}$) and two at either side of the concrete wedge ($A_{side}$).

![Figure 6-13: Assumed failure surfaces of concrete wedge and assumed stress distribution at time of failure](image)

The stress distribution across the concrete wedge surface at the time of failure is unknown. In concrete failure cones of fasteners subject to tensile forces, which experience failure surfaces comparable to fasteners located close to a free concrete edge and subject to lateral forces, the tensile stress distribution across the cone-shaped failure surface, although irregular and variable for different cone sizes, was
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found to be reasonably approximated by assuming a triangular shape (Zhao 1995). The position of the peak tensile stress moves towards the free edge as the cracks propagate under increasing loads. At a critical crack length \( e_{\text{crit}} \) for the peak tensile stress, cone break-out occurs. In fastener applications, the ratio of the critical length in regards to the overall length \( e_{\text{crit}}/e \) was found to vary between 35%–55% whereby the ratio decreases with an increasing overall length which can be attributed to the ‘size effect’ in concrete.

Assuming a similar triangular stress distribution acting on the individual failure surfaces of the concrete wedge where the peak stress equals the concrete tensile strength \( f_t \) at the position \( e_{\text{crit}}/e \) as shown in Figure 6-13, the sum of the projected horizontal force components of the individual failure surfaces amount to its overall strength:

\[
\begin{align*}
P_{\text{wed}} = & \int \int \sigma(h) \, dA + 2 \int \int \sigma(e) \, dA \\
\end{align*}
\]

where

\[
\begin{align*}
\int \int \sigma(h) \, dA = & \frac{1}{2} f_t h_r \left[ s_x + d_{bs} + \frac{2}{3} \tan(\alpha) \left( 1 + \frac{e_{\text{crit}}}{e} \right) \right] \\
\end{align*}
\]

and

\[
\begin{align*}
\int \int \sigma(e) \, dA = & \frac{1}{6} f_t h_r \frac{e_h}{\tan(\alpha)} \left( 1 + \frac{e_{\text{crit}}}{e} \right) \\
\end{align*}
\]

From the push-out test results, the factor \( e_{\text{crit}}/e \) for the concrete rib geometries investigated was determined to be 0.35 for shear connectors placed in the central or favourable position of the concrete rib, a value which is consistent with that found for fastener applications (Zhao 1995). In this calibration, the concrete tensile strength \( f_t \) was taken as \( 0.5 f_{\text{cm}}^{0.5} \). Substituting Equations (6-12) and (6-13) and the constant \( e_{\text{crit}}/e \) into Equation (6-11) yields the capacity of the concrete wedge against break-out failure to be

\[
\begin{align*}
P_{\text{wed}} = & \frac{1}{2} f_t h_r \left[ s_x + d_{bs} + 0.9 b_{0,\text{side}} \right], \\
\end{align*}
\]

with
Design models for secondary beam stud connections

\[ b_{0,side} = \frac{e_t + e_b}{\tan(\alpha)} = \frac{2e}{\tan(\alpha)}. \]  (6-15)

For single studs, or double studs where the individual concrete failure surfaces do not overlap, i.e. \( s_x > 2e/\tan(\alpha) \), the capacity of the concrete wedge, or wedges, can be calculated from Equation (6-14) setting \( s_x \) to zero.

### 6.3.2.4 Application of a stud performance-enhancing device

If a stud performance-enhancing device is used, it influences the rib punch-through behaviour in two ways. It increases the effective diameter at the base of the stud from where the diagonal concrete cracks start to propagate. Hence, in Equation (6-14) the stud diameter is replaced by the diameter of the enhancing device \( d_{ed} \), i.e.

\[ d_{hs} = d_{ed}. \]  (6-16)

At the same time, the device increases the effective height of the shear connector over which the horizontal shear forces are transferred into the concrete as the base of the stud is effectively stiffened over the full height of the device, \( h_{ed} \). The effective height in Equation (6-7) is consequently determined as

\[ h_{ec} = h_{ed}. \]  (6-17)

### 6.3.2.5 Transverse bottom reinforcement in concrete rib

Where sufficient transverse bottom reinforcement is provided in the bearing zone of the shear connector in the concrete rib to suppress the effects of longitudinal splitting failure, whether it be plain transverse reinforcement bars or transverse bars as part of the waveform reinforcement element, the concrete rib capacity is increased (see Chapter 4.5.2.7). However, as this effect was only investigated on shear connections comprising of 19mm diameter stud connectors, the beneficial effects should be initially restricted to these types of applications. It was found that a transverse bottom reinforcement diameter of \( d_{br}=6\)mm, placed at around mid-height of the
Design models for secondary beam stud connections

concrete rib was sufficient to increase the rib punch-through strength by at least 20%. Hence,

\[ P_{\text{wed,bs}} = k_{bs} P_{\text{wed}} \]  \hfill (6-18)

with

\[ k_{bs} = \begin{cases} 
1.2 & \text{for } d_{bs}=19\text{mm and } d_{br}\geq6\text{mm}, \\
1.0 & \text{for } d_{bs}\geq19\text{mm}.
\end{cases} \]

The transverse reinforcement bar should be placed as low as possible in the concrete rib and a minimum of 40mm in front of the stud in the region where the rib punch-through wedge forms. If this direction is not known, a bar should be placed on both sides of the stud as is the case for the waveform component. However, as most modern types of steel decking consist of lap joints or stiffeners in the base of a concrete rib, the reinforcing mesh containing the transverse bars can be placed on top of these elements.

6.3.2.6 Force component transferred by steel decking

The load-transfer mechanisms of the steel decking which are thought to increase the shear connection strength when rib punch-through failures are experienced are shown in Figure 6-14. Once the concrete wedge in the bearing zone of the shear connection starts to develop, the steel decking restricts the longitudinal movement of the failure wedge and ensures an additional load transfer \( P_{sh} \) which is characterized by the bulging of the steel decking (see also Figure 4-34 and Figure 4-35). The upper and lower edges of the steel decking are considered to serve as stiff horizontal supports. At the top edge, a compressive force \( C_1 \) develops which is transferred into the surrounding concrete provided the steel decking does not separate from the concrete slab. At the bottom corner, a tensile force develops in the steel decking \( T_{sh} \) which, at the rear side of the shear connection, is anchored into the concrete rib \( C_2 \) (Figure 6-14a). Moreover, some load transfer would take place in the transverse direction of the steel decking across the width of the concrete rib. The strengthening effect of the steel decking ceases eventually when, under very large deformation, the
Design models for secondary beam stud connections

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tensile force $T_{sh}$ exceeds the tensile capacity of the steel decking and the concrete is able to punch-through the fractured steel decking (Figure 4-35).

![Diagram](image)

**Figure 6-14:** Model for load transfer mechanisms of steel decking in shear connections experiencing rib punch-through failure

After cracking of the concrete wedge, the forces applied to the steel decking can be divided into a component acting normal to the steel decking surface $p_{sh}$ and a longitudinal shear component, $\tau_{sh}$ (Figure 6-14b). As the angle of the steel decking to the vertical, $\gamma$, remained relatively small for the steel decking geometries investigated, any effects due to the shear force component where generally ignored. Furthermore, if any load transfer in the transverse steel decking direction is neglected, the normal force component initially transferred by the steel decking via bending $p_{sh,b,I}$ can be determined as a function of its mid-span deformation from the bending mechanism I shown in Figure 6-14c where

$$p_{sh,b,I} = \frac{32}{\ell^4} \frac{E_{sh}}{b_{sh} t_{sh}^3} \delta_{sh} \leq \frac{12}{\ell} m_{pl}, \quad (6-19)$$

with
Design models for secondary beam stud connections

\[ m_{pl} = f_{y,sh} S_{sh,pl} = f_{y,sh} \frac{b_{esh} t^2}{4} \]  

and

\[ \ell = \sqrt{h^2 + (e_s - e_h)^2}. \]

The effective width of the steel decking \( b_{esh} \) is assumed to be similar to the width of the concrete wedge at the base of the concrete rib:

\[ b_{e,sh} = \frac{2e_h}{\tan(\alpha)} + d_{bs} + s_x = b_{b,side} + d_{bs} + s_x. \]

Once the first set of plastic hinges forms at both supports of the steel rib, the bending stiffness is reduced and any further increase in bending capacity is determined in accordance with the bending mechanism II of Figure 6-14c:

\[ p_{sh,ill} = \frac{32}{5} \frac{E_{sh} b_{esh} t^3}{\ell^4} \left( \delta_{sh} - \delta_{sh,I,max} \right) + \frac{12m_{pl}}{\ell^2} \leq \frac{16m_{pl}}{\ell^2}, \]

where \( \delta_{sh,I,max} \) is the corresponding mid-span deformation of the steel decking at which the plastic hinges are assumed to form in mechanism I. If the maximum bending capacity of mechanism II is reached, a plastic hinge forms at mid-span creating the kinematic mechanism III where no further increase in bending capacity is possible but continuous load transfer at the maximum bending capacity of

\[ p_{sh,ill} = \frac{16m_{pl}}{\ell^2}, \]

is still ensured under increasing deformations.

Where the deformations of the steel decking become larger, the tensile force mechanism shown in Figure 6-14d ensures an additional load transfer of the steel decking. The load capacity \( p_{sh,t} \) of this mechanism can be determined using force equilibrium to be

\[ p_{sh,t} = \frac{2T_{sh} \sin \theta}{\ell}, \]

where, from geometry,

\[ \sin \theta = \frac{2\delta_{sh}}{\sqrt{4\delta_{sh}^2 + 0.25\ell^2}}. \]
The reaction force in the steel decking $T_{sh}$ is obtained by assuming linear-elastic material behaviour up to its yield strength $f_{ysh}$:

$$T_{sh} = \frac{\Delta \ell}{\ell} E_{sh} b_{esh} t \leq T_{ysh} = f_{ysh} b_{esh} t$$  \hspace{1cm} (6-27)

where the elongation $\Delta \ell$ equals the length of the sector of the circle shown in Figure 6-14d minus the chord length $\ell$:

$$\Delta \ell = \ell \left( \frac{\pi}{2} \frac{2 \theta^*}{\sin 180} - 1 \right).$$  \hspace{1cm} (6-28)

If it is now assumed that the horizontal component of the sheeting deformation, $\delta_{sh \cos(\gamma)}$, is of the same magnitude as the slip experienced by the shear connection $\delta_c$, the additional force component transferred by the steel decking $P_{sh}$ can be determined at any given slip as the addition of the individual bending and tensile components to be

$$P_{sh} (\delta_c) = p_{sh,b} (\delta_c) + p_{sh,t} (\delta_c).$$  \hspace{1cm} (6-29)

It should be noted that for large deformations where the tensile forces in the steel sheeting start to approach the yield strength, the resistance of the steel due to bending is reduced. In the limit, it is only the tensile yield that can provide horizontal resistance, hence

$$P_{sh} (\delta_c) \rightarrow p_{sh,t} (\delta_c) \ell \quad \text{for} \quad T_{sh} \rightarrow T_{ysh}.$$  \hspace{1cm} (6-30)

As part of the investigation into the shear connection behaviour described in Chapter 4, specimens which had the steel decking omitted but consisted of an otherwise similar stud arrangement and material properties were tested in W-dek® concrete rib geometry. At any given slip, the difference in applied horizontal force between similar specimens with and without decking can be considered as the steel decking component of the force. Hence, the force differences at the various slip measurements of these set of tests are compared with the theoretical steel decking components $P_{sh}$ in Figure 6-15 for single stud arrangements and a sheeting thickness of $t=0.75\text{mm}$ and in Figure 6-16 for a stud pair arrangement and a thickness of $t=1.00\text{mm}$ using the measured material properties.
It can be seen that at lower slips, the proposed provisions for the decking force component seems to provide a conservative estimate whereas at larger slips, the theoretical tensile force component seems to increase disproportionately. The tests results indicate that at larger deformations starting at around 3-6mm slip, the steel decking component does not increase any further. This behaviour may be attributed to the crushing of the concrete in the bearing zone between the connector and steel decking which prevents an increase in shear connector slip being translated into any further deformation of the steel decking. The loss in strength of the crushed concrete would also restrict the amount of force which is able to be transferred to the steel decking. Based on the test results, it was therefore decided to limit the increase in the steel decking component to a shear connection slip of 3mm, hence

\[ P_{sh} \left( \delta_c \right) \leq P_{sh} \left( \delta_c = 3\text{mm} \right). \] (6-31)

It is interesting to note that this limit closely corresponds to the point of yielding in the sheeting. The limitation seems to provide a reasonable estimate for this force component at the ultimate slip capacity of the specimens which, in accordance with Eurocode 4 (CEN 2004), is defined as \( \delta_{uc}=6\text{mm} \) slip. As can also be seen, the bending component is relatively small at larger slips and could be neglected for design which would then automatically account for the relation, given in Equation (6-30), for tensile stresses approaching the sheeting yield stress.
6.3.2.7 Rib punch-through capacity

The maximum capacity per stud $P_{RPT,\text{max}}$ against rib punch-through failure is determined by adding the capacities of the plain concrete and the force component provided by the steel decking at maximum load. For the shear connections investigated, the maximum load at which rib punch-through failure occurred was experienced at around 2mm slip. By rearranging Equation (6-7) and taking into account the number of shear connectors per wedge, $n_x$, the maximum strength per stud connector is determined as

$$P_{RPT,\text{max}} = \frac{1}{n_x} \left[ \left( \frac{h_{ce} + 1.5e_c}{h_c} \right) P_{\text{wed}} + P_{sh} \left( \delta_c = 2\text{mm} \right) \right]$$

(6-32)

where $P_{\text{wed}}$ is determined in accordance with Equation (6-14) or (6-18) whichever applicable and $P_{sh}$ is given by Equation (6-29).

For shear connections which do not include the waveform component, the minimum rib punch-through capacity per stud which can be guaranteed at any given slip up to the required slip capacity is also required (see Figure 6-10). Here, the force component of the steel decking experienced at minimum load which was found to occur at slips larger than 3mm can be considered. The sum of the concrete component of the minimum load per shear connector, derived from Equation (6-8), and the decking component, derived from Equation (6-31), provides the total minimum capacity:

$$P_{RPT,\text{min}} = \frac{1}{n_x} \left[ \left( \frac{h_{ce} + 1.5e_c}{h_c} \right) - 1 \right] P_{\text{wed}} + P_{sh} \left( \delta_c = 3\text{mm} \right)$$

(6-33)

In Figure 6-17, the maximum strength of shear connections which experienced rib punch-through failure are compared to the strength prediction in accordance with Equation (6-32). Where the KF70® geometry was used, the internal beam test results given in Oehlers and Lucas (2001) and the test results of Patrick (2002c) were also taken into consideration where applicable. Where available, the measured dimensions and material properties were used in the strength prediction. Otherwise, the calculated means of the variables in accordance with Chapter 8 were employed. The strength provision seems to provide a reasonable accurate estimate of the
connection strength for both conventionally reinforced specimens and specimens incorporating the reinforcing devices. The scatter is suspected to originate in the activation of the tensile stresses of the concrete which itself is a highly variable parameter. Also, the size and shape of the failed concrete wedge was found to be subject to some variation.

![Graph showing comparison of maximum shear connection strength with predicted mean strength $P_{\text{RPT,max}}$ for specimens experiencing rib punch-through failure](image)

**Figure 6-17:** Comparison of maximum shear connection strength with predicted mean strength $P_{\text{RPT,max}}$ for specimens experiencing rib punch-through failure

For the determination of the minimum allowable shear connection strength $P_{\text{RPT,min}}$ given by Equation (6-33), the bending capacity of the steel decking $P_{\text{sh,b}}$ was generally ignored in accordance with the relation given in Equation (6-30) as most of the steel deckings were assumed to have experienced stresses in the vicinity of their yield stress. In Figure 6-18, the comparison of the minimum rib punch-through strength with the specimen strength at the required slip capacity, which is set to 6mm slip in accordance with Appendix B of Eurocode 4 (CEN 2004), verifies the conservative strength estimate that the provision gives as most test results are well above the predicted strength but with relatively little scatter.
6.3.3 Rib shearing and stud pull-out failures

6.3.3.1 General

In Chapter 4.5.2, it was shown that both failure modes are closely related and whether rib shearing or stud pull-out failure occurred solely depended on the width of the concrete slab. It was further shown that both failure modes are initiated by a horizontal cracking between the concrete rib and the cover slab which originates at the rear side of the concrete rib. The failure surface was found to then propagate across the concrete rib in both the longitudinal and transverse directions while in its longitudinal direction, it locally passed over the head of the stud. The rib shearing approach by Patrick and Bridge (2002a), which was presented in Chapter 2.3.2.3, takes this behaviour into account by defining the rib shearing capacity as the point when the vertical tensile stress at the rear of the concrete rib from rib rotation exceeds the tensile capacity $f_t$ of the concrete acting over an effective width $b_{eff}$.

Based on Equation (2-23), a refined approach for both rib shearing and stud pull-out is proposed:
Design models for secondary beam stud connections

\[ P_{RS/SP} = \frac{1}{6} k_{ec} b_{eff} \left( \frac{b_r}{n} \right)^2 k_{\sigma} f_t \] (6-34)

where \( k_{ec} \) is a correction factor taking the embedment depth of the stud into account and \( k_{\sigma} \) is a reduction factor considering the non-linear stress distribution across the failure surface. Both factors are explained in the following. The geometry of the concrete rib is defined by its height, \( h_r \), and its width at the top of the rib, \( b_r \).

### 6.3.3.2 Effective width of failure surface

It is thought that the bending related vertical stresses initially develop over a similar effective width as the stresses leading to the break-out of the concrete wedge (as shown in Figure 6-13). Where the width of the concrete flange \( b_{cf} \) is smaller than the effective width, the transverse distribution of the vertical stresses is defined by the concrete slab geometry and a rib shearing failure can be expected. However, rib shearing failures were also observed in tests where the concrete slab was wider than the effective width. As concrete cracks will generally follow the path of least resistance, it can be assumed that if the width of the expected final concrete cone failure surface \( b_{cone} \) for stud pull-out failure is smaller than the concrete flange width, \( b_{cf} \), a rib shearing failure will also occur.

The widths of the final failure cones for specimens experiencing stud pull-out failures were measured for some of the W-Dek® geometry specimens and were found to be subject to large variation. However, on average, the failure cone was found to propagate around 300mm in the transverse direction from either side of the head of the stud which resembles an inclination of about 1:2.5. A typical failure cone is shown in Figure 4-44. As no measurements of the failure cones in the tests incorporating the KF70® geometry have been undertaken, the failure cone width is estimated accordingly as

\[ b_{cone} \approx 5h_c + s_x \] (6-35)

where \( s_x \) is the transverse centre-to-centre stud spacing.
The masonite strain gauge measurements taken at the rear side of the concrete ribs in push-out tests (as for example shown in Figure 4-41) indicated that the transverse stress distribution across the width of the concrete slab was not uniform for the specimens experiencing rib shearing failures. Therefore, the effective width over which a uniform stress distribution is assumed must be less than the total slab width. For shear connections subject to rib shearing failures, only about 90% of the strength of similar connections in wider concrete slabs subject to stud pull-out failures were reached, even where the width of the specimens was wider than the predicted failure cone width (see Chapter 4.5.2.6). Hence, the effective width for specimens experiencing rib shearing failures is reduced accordingly. The effective width for the two failure modes can be determined as

\[
\begin{align*}
    b_{\text{eff,SP}} &= b_{\text{est}} \\
    b_{\text{eff,RS}} &= 0.9 \min(b_{\text{est}}, b_{cf}) \quad \text{for} \quad \begin{cases} b_{\text{cone}} \geq b_{cf} \\ b_{\text{cone}} < b_{cf} \end{cases}
\end{align*}
\]

(6-36)

where \(b_{\text{est}}\) is the width of the assumed wedge (see Figure 6-13):

\[
b_{\text{est}} = \frac{2e_i}{\tan(\alpha)} + d_{bs} + s_x = b_{t,\text{side}} + d_{bs} + s_x.
\]

(6-37)

If the width of the concrete slab is eccentric, e.g. the connection is close to one free edge, as for example in edge beam applications, the two effective widths determined in accordance with Equation (6-36) can be averaged.

### 6.3.3.3 Correction factor \(k_{ec}\) for stud embedment depth

Both stud pull-out and rib shearing failures are characterized by the failure surface locally passing over the head of the stud. Therefore, the embedment depth of the shear connector into the concrete cover slab has a profound impact on the failure surface in the vicinity of the shear connectors. In contrast, Equation (6-34) assumes a horizontal failure surface across the complete concrete rib \(A_{\text{straight}}\). The correction factor \(k_{ec}\) is thought to account for the differences in the assumed and the actual failure surface \(A_{\text{cone}}\):

\[
k_{ec} = \frac{A_{\text{cone}}}{A_{\text{straight}}} = \frac{b_{r,\text{cone}} b_{\text{eff,cone}}}{b_{rt} b_{\text{eff}}} + \left(1 - \frac{b_{\text{eff,cone}}}{b_{\text{eff}}}\right)
\]

(6-38)
The ratio $b_{r,cone}/b_{rt}$ is the increase in length of the failure surface in longitudinal direction and $b_{eff,cone}/b_{eff}$ is the width over which this increase is assumed to appear (see Figure 6-19). In tests, it was observed that the cone failure surface at the rear side of the concrete rib propagates at an angle of approximately 45° transversely towards the top edge of the steel decking. Similar observations of the failure cone surface were also made in Lloyd and Wright (1990). The two ratios can then be determined to:

$$\frac{b_{r,cone}}{b_{rt}} = \sqrt{(b_{r} - e_{i})^2 + (h_{c} - h_{r})^2} \frac{b_{rt}}{b_{rt}},$$

(6-39)

$$\frac{b_{eff,cone}}{b_{eff}} \approx \frac{(b_{r} - e_{i}) + d_{rs} + s}{b_{eff}} \leq 1.0.$$

(6-40)

**Figure 6-19:** Idealized failure surface of rib shearing / stud pull-out failures

### 6.3.3.4 Stress distribution along failure surface

In Patrick and Bridge (2002a), a linear stress distribution along the failure surface was assumed (see also Figure 2-12) which implies that the concrete rib cross-section remains plane. However, for the concrete rib geometries tested, the height to length
ratio of the stocky cantilever was generally in excess of 2.0 so that this assumption is no longer valid. The system can rather be compared to an unreinforced concrete corbel. Measured horizontal stress distributions along the corbel-column interface for various unreinforced corbel geometries are shown in Figure 6-20. It can be seen that the actual stress distribution differs significantly from the linear distribution assumed in normal beam theory. Generally, the stresses are more concentrated in the upper and lower regions of the corbel which results in a larger inner lever arm but in a reduced stress resultant. Hence, the moment capacity for a given peak stress compared to that for a cantilever application is reduced. This effect is stronger the larger the height to length ratio becomes. However, the point of zero stress remains essentially at mid-span. From Figure 6-20, it can also be seen that the influence of the loading point on the initial tensile stress distribution is negligible as is the influence of the corbel shape (inclined or rectangular shape). Therefore, it is deemed to be justified to assume that a similar initial tensile stress distribution also exists at the concrete rib–cover slab interface in a secondary beam shear connection.

Figure 6-20: Horizontal stress distributions at various corbel-column interfaces (obtained from test results published in Mehmel and Becker (1965))
In Chapter 4.5.2.10, it was shown that after the first crack appears at the rear side of the specimen, a further increase in load can be accommodated as the crack propagates longitudinally and transversely across the beam. This could possibly converge the stress distribution more towards a linear distribution and increase the stress resultant while reducing the inner lever arm. To obtain the exact stress distribution at failure, a detailed FE-analysis of the shear connection investigated would need to be performed. However, as a simplification, a reduction factor  \( k_\sigma \) is introduced which takes into account the difference in moment capacity compared to that for the linear beam capacity distribution along the concrete rib-cover slab interface. For the trapezoidal rib geometries investigated, this factor was found to best correlate with test results as

\[
k_\sigma = \frac{\int \sigma_y(x) x dx}{1/6 \sigma_{y,max} b_{rt}^2} = 0.85.
\]

(6-41)

In Figure 6-20, a proposed stress distribution is shown which would reflect this reduction factor. The factor seems to provide a feasible assumption as the stress distribution correlates well with the measured stress distributions of the concrete corbels, in particular with the ones consisting of the larger height to length ratios \( (h_{co}/a_{co}) \).

### 6.3.3.5 Application of the waveform reinforcement element

Where the waveform reinforcement element Deckmesh® was applied to shear connections incorporating the concrete rib geometries investigated, rib shearing and stud pull-out failures were successfully suppressed where the element was sufficiently anchored in the concrete cover slab, i.e. to the top face reinforcement. Hence, if this specific element is provided in the current Australian types of profiled steel decking, the failure modes rib shearing and stud pull-out no longer need to be considered. However, where other types of reinforcement solutions or rib geometries are used, the strength of the shear connection can be determined in a manner similar to that for a reinforced concrete corbel application. Figure 6-21 gives a simple strut- and tie model in the style of the models used for the design of reinforced concrete corbels.
If it is ensured that the vertical reinforcement bars placed in the concrete rib can
develop their full tensile capacity $f_{sy}$ across the failure surface, the shear connection
strength can be determined as

$$P_{RS/SP} = \frac{T_z z \sin \gamma_{sw}}{n_z h_r} = \frac{f_{sy} A_{sw} 0.9(b_n - c_b)}{n_b h_r} \sin \gamma_{sw}$$  \quad (6-42)

where $c_b$ is the horizontal distance of the bars from the edge of the concrete rib, $A_{sw}$ is
the total area of the waveform reinforcement crossing the failure surface within the
effective width $b_{eff}$ and $\gamma_{sw}$ is the inclination of its reinforcement bars towards the
vertical line. Note that the strength provided by the reinforcement should be greater
than that for the unreinforced concrete rib given by Equation (6-34).

As the variation of the waveform reinforcement geometry was not part of the
experimental investigations described in Chapter 4.3, the vertical reinforcing bars
should be detailed in accordance with the provisions given for Type 4 longitudinal
shear reinforcement in AS2327.1 (Standards Australia 2003a), e.g. the concrete rib
must be reinforced over at least 400mm width whereas the spacing between the
individual bars should not be greater than 150mm (see also Figure 2-20). However, it
should be noted that the waveform reinforcement must include transverse bars to
prevent pull-out failures between the longitudinal bars (see Chapter 4.5.2.9).
6.3.3.6 Assessment of rib shearing and stud pull-out capacity

The maximum strength of push-out test results for the Australian types of trapezoidal steel decking which experienced either stud pull-out or rib shearing failures are compared with the predicted shear connection capacity of Equation (6-34) in Figure 6-22. The provisions seem to provide a satisfactory estimate of the experienced connection strengths and the scatter is significantly reduced compared to previous approaches. A further reduction in scatter might not be achievable as the two failure modes are strongly influenced by the tensile strength of the concrete which in itself is a variable which can experience large variation.

![Comparison of maximum shear connection strength with predicted strength](image)

**Figure 6-22:** Comparison of maximum shear connection strength with predicted strength $P_{SP/RS}$ for specimens experiencing rib shearing or stud pull-out failures

6.3.4 Simplified method for Australian trapezoidal steel decking geometries

As the new design method proposed is rather complex to use, and as reduction factors of the solid slab shear connections strength provide by far the most convenient approach to take into account the influence of steel decking, such reduction factors have been derived by applying the new proposed method to the
most common applications of the Australian types of trapezoidal steel decking and comparing the results to the solid slab provisions as given by Equations (6-1) and (6-2). The shear capacity of a headed stud of 19mm diameter in a secondary beam application $P_{\text{simp},m}$ can then be determined as

$$P_{\text{simp},m} = k_{t,m} P_{\text{solid}}$$  (6-43)

where $k_{t,m}$ is the reduction factor given in Table 6-6.

**Table 6-6**: Reduction factors $k_{t,m}$ for secondary composite beam applications for the determination of the expected mean stud capacity in accordance with Equation (6-43)

<table>
<thead>
<tr>
<th>Geometry</th>
<th>Number of studs $n_x$</th>
<th>Concrete compressive strength $f'_c$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$&lt;$ 32MPa</td>
</tr>
<tr>
<td></td>
<td></td>
<td>CR$^1$</td>
</tr>
<tr>
<td>KF70$^6$</td>
<td>1</td>
<td>N/A</td>
</tr>
<tr>
<td>W-Dek$^8$</td>
<td>1</td>
<td>N/A</td>
</tr>
<tr>
<td>KF70$^6$</td>
<td>2</td>
<td>N/A</td>
</tr>
<tr>
<td>W-Dek$^8$</td>
<td>2</td>
<td>N/A</td>
</tr>
</tbody>
</table>

$^1$: CR: conventional reinforced specimens, $^2$: ED: stud enhancing device, $^3$: WR: waveform reinforcement element $^4$: only applicable for sheeting thickness of $t \leq 0.75$mm, otherwise N/A $^5$: only applicable for central positioned studs with a height of $h_c \geq 150$mm, otherwise N/A

The values given in Table 6-6 are applicable for the use of studs which extend at least 40mm above the top of the ribs of the profiled steel decking, hence $h_c \geq 100$mm for the KF70$^6$ geometry and $h_c \geq 120$mm for the W-Dek$^8$ geometry, unless stated otherwise. Preferably, the studs should be placed in the central position of the pan which can be ensured by pre-holing the steel decking as has been common practice in several European countries for years, e.g. Germany. This would also overcome the quality concerns of some of the stud welds when fast welded through the steel decking (see Chapter 7.2). Where pairs of studs are used, a diagonal lay-out could alternatively be applied where the studs are placed on either side of the lap joint or stiffener. In any case, the minimum clear distance between the head of the studs in transverse direction, as given in Clause 8.4.2 of AS2327.1 (Standards Australia 2003a), must be increased to 2.5 times its shank diameter. Where single studs in combination with the waveform element are used, the studs could also be placed on the favourable side of the lap joint or stiffener as the effects of rib shearing or stud
pull-out failures are suppressed. However, the practicability and quality control of such a shear connector design on-site might prove to be a problem. If the single studs are placed in alternating positions on either side of the lap joint, as suggested by Eurocode 4 (CEN 2004), the reduction factors of Table 6-6 should not be applied as the shear connectors positioned on the unfavourable side might experience a significantly reduced strength or slip capacity. The values given in Table 6-6 were derived for high strength G550 sheeting material with nominal thicknesses ranging from 0.6mm to 1.00mm.

The Type 4 longitudinal shear reinforcement provisions of Chapter 9.8 of AS2327.1 (Standards Australia 2003a) additionally need be considered where the shear connection is close to a free concrete edge in transverse direction. They are generally deemed to be satisfied where the waveform element is provided. In all other cases where the reduction factors of Table 6-6 are applied, additional Type 4 shear reinforcement needs to be provided where the distance of the transverse edge of the concrete slab to the nearest shear connector is less than 2.5 times the height of the shear connector.
7 COMPOSITE BEAM TESTS

7.1 General

The composite beam tests described in this chapter were carried out in order to directly compare the findings of the shear connection strengths and slip capacities obtained from earlier small-scale push-out tests (see Chapter 4) with a full-scale beam application. The main aspects considered in the design of the tests were the following:

- Do the concrete-related brittle failure modes experienced in push-out tests also appear in full-scale composite beams if the existing design provisions of a major international standards, namely Eurocode 4 (CEN 2004), are fulfilled?
- How does brittle shear connection behaviour affect the overall beam performance?
- Is the application of the novel reinforcing elements a suitable measure to suppress the brittle failure modes and increase strength and ductility of the shear connection in a full-scale beam application?
- How do the shear connection strengths and deformation capacities of the full-scale beam tests compare to the results from small-scale push-out tests and hence, compare to the new design method given in Chapter 6.3?

Apart from these main aspects, other valuable information about the behaviour of the shear connection, including the service load behaviour and the effect of flexural stiffness of composite beams with low degrees of shear connection, was also provided by the composite beam tests.
Due to financial and time restraints, it was decided to concentrate the composite beam investigation on secondary beams as this is the most common application. The tests on two full-scale beams, one comprising an internal beam stud application and one an edge beam stud application are described in the following. Each beam was divided into two halves where one half was conventionally reinforced while the other half included the novel reinforcing components required to ensure a sufficiently ductile shear connection in accordance with the design provisions given in Chapter 6. The splitting of the composite beams in two halves provided the advantages of a direct comparison of the shear connection behaviour with different layouts with all material properties remaining identical. It was also possible to double the number of variables to be investigated for the given number of tests. This practice has already been successfully employed in the past (Patrick et al. 1995). However, the existence of two shear connections with different strengths and deformation capacities on either side of the same beam required special test procedures and reinforcing measures as described in Chapter 7.4, to prevent failures of the composite beam before both connections had been tested to a sufficient level of slip.

Along with the composite beam tests, companion push-out tests were performed for a direct comparison of the test results. The accompanying push-out tests have already been described in detail in Chapter 4 (CTR-RIB5 series). Each composite beam specimen was equipped with numerous strain gauges across various cross-sections in order to obtain detailed information about the shear connection behaviour. With the help of these strain gauge measurements, the shear connection forces could be accurately determined and compared with the results of the push-out tests as described in Chapter 7.6.

### 7.2 Design and construction of test specimens

It was decided to test comparatively short beams in order to keep the individual beam costs low, but to still provide a sufficient number of shear connectors in each shear span to reflect the force distribution between individual connections in a full-scale composite beam. As the focus of this investigation was the determination of the
strength and behaviour of the shear connection, partial shear connection needed to be provided in the shear spans tested whereby the degree of shear connection was such that it fulfilled the minimum requirements given in Eurocode 4 (CEN 2004). A single load was deemed to be the favourable option as it provided the longest possible shear span without the local influence of applied loads; this allowing for a simpler investigation of the redistribution of the stud forces between the individual shear connections. The two composite beam specimens were chosen and their degree of shear connection when subject to a single load applied at the designated loading point for the design strength were determined in accordance with Eurocode 4 (CEN 2004). The two beam specimens are summarized in Table 7-1, where the expected shear connection strength determined in accordance with the design method suggested in Chapter 6 is also given. Further details of the two beams are shown in Figure 7-1 and Figure 7-2.

Both test specimens comprised a 250UB31.4 steel beam section with a 150mm deep composite slab. For the internal beam application (Beam 1), the concrete slab was chosen to be 1800mm wide whereas for the edge beam application (Beam 2), the concrete slab had an outstand of 300mm from the centreline of the shear connection to the closest edge providing an overall concrete slab width of 1200mm. It was decided to use W-Dek® steel decking of 0.75mm thickness for the composite beam tests as the more systematic small-scale push-out test investigation was also performed on this type of rib geometry.

The application of 19mm diameter stud pairs ensured that the largest possible variety of secondary beam failure modes could be expected in the very limited number of composite beam tests (see Table 7-1). The studs were of a specified height of 127mm after welding and placed at 80mm transverse centres in order to fulfill the requirements of Eurocode 4 (CEN 2004). The effective spans of both beams were set to 4200mm between the end supports. In order to be able to load the beams up to the predicted shear connection strength of its stronger side without failing the far side with the weaker shear connections, two extra pairs of shear connectors were required between the two individual loading points (see Figure 7-1 and Figure 7-2). This resulted in a total number of 10 stud connectors placed in each shear span tested as it was deemed beneficial to have identical shear spans on both sides of each beam. The
nominal strength grade of the steel beam was grade 300 (OneSteel 300PLUS®) and the concrete target strength for both concrete slabs was set to 32MPa.

### Table 7-1: Summary composite beam test specimens

<table>
<thead>
<tr>
<th>Application</th>
<th>Beam 1</th>
<th>Beam 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel section</td>
<td>250UB31.4</td>
<td>250UB31.4</td>
</tr>
<tr>
<td>Beam span (L)</td>
<td>4200mm</td>
<td>4200mm</td>
</tr>
<tr>
<td>Concrete slab dimension</td>
<td>$b_{cf}=1800\text{mm}, D_{c}=150\text{mm}$</td>
<td>$b_{cf}=1200\text{mm}, D_{c}=150\text{mm}$</td>
</tr>
<tr>
<td>Steel decking</td>
<td>W-Dek® ($t=0.75\text{mm}$)</td>
<td>W-Dek® ($t=0.75\text{mm}$)</td>
</tr>
<tr>
<td>Stud dimensions</td>
<td>$d_{mm}=19\text{mm}, h_{c}=127\text{mm}$</td>
<td>$d_{mm}=19\text{mm}, h_{c}=127\text{mm}$</td>
</tr>
<tr>
<td>Studs per concrete rib ($n_x$)</td>
<td>2 @ 80mm centres$^2$</td>
<td>2 @ 80mm centres$^2$</td>
</tr>
<tr>
<td>Target concrete strength ($f'_{c}$)</td>
<td>32MPa</td>
<td>32MPa</td>
</tr>
<tr>
<td>Mesh reinforcement</td>
<td>Side A (weak side) 2xSL72$^1$</td>
<td>Side B (strong side) SL72</td>
</tr>
<tr>
<td>Spiral enhancing device</td>
<td>-</td>
<td>X</td>
</tr>
<tr>
<td>Number of studs in shear span</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>Design stud strength$^4 (f_{d,EC4})$</td>
<td>57kN</td>
<td>57kN</td>
</tr>
<tr>
<td>Degree of shear connection for design stud strength ($\beta_{dm}$)</td>
<td>0.50</td>
<td>0.50</td>
</tr>
<tr>
<td>Predicted stud strength$^5 (f_{vs})$</td>
<td>57kN</td>
<td>83kN</td>
</tr>
<tr>
<td>Degree of shear connection for predicted stud strength ($\beta_{pm}$)</td>
<td>0.45</td>
<td>0.66</td>
</tr>
<tr>
<td>Predicted failure mode$^4$</td>
<td>Stud pull-out</td>
<td>Rib punch-through</td>
</tr>
</tbody>
</table>

$^1$: Slab outstand 300mm from the long. centreline of the shear connection.

$^2$: Stud placed in a diagonal position 35mm to either side of the lap joint or pan stiffener.

$^3$: Top and bottom reinforcement.

$^4$: Design strength in accordance with Eurocode 4 (CEN 2004) using predicted material properties.

$^5$: Determined using the general design provisions given in Chapter 6.3.
Initially each shear connector was welded directly through the profiled steel decking in a diagonal position 35mm to either side of the lap joint or pan stiffener as is usually done in practice. However, after the welding process was completed, it was observed that approximately half the studs did not have a 360 degree weld bead (Figure 7-3). When the studs were tested in bending by using a hammer, around 25% of the studs failed and it could be clearly seen that the weld had not penetrated satisfactorily between stud shank and steel section (Figure 7-4). AS/NZS1554.2 (Standards Australia 2003b) requires the studs which do not have a 360 degree weld bead to be either replaced or repaired by adding a minimum fillet weld. Due to the large number of unsatisfactory stud welds and due to the bad quality of nearly all the welds, it was decided to remove all studs from the top flange, pre-hole the steel decking and weld the new studs directly to the repaired steel beam top flange. However, the pre-holing of the steel decking reduces the predicted design strength given in Table 7-1 to $f_{d,e,EC4} = 49\text{kN}$ for all specimens which subsequently reduces the degree of shear connection, considering the design stud strength at maximum

Figure 7-1: Beam 1 – construction details
bending, to $\beta_{dm} = 0.43$ for both beams. The reduced degree of shear connection still fulfils the minimum shear connection requirements given in Eurocode 4 (CEN 2004) but no longer the requirements of AS2327.1 (Standards Australia 2003a). However, the pre-holing of the steel decking is not expected to have a major effect on the expected shear connection capacity and the predicted stud strengths using the provisions given in Chapter 6.3 which therefore remain unaltered.

**Figure 7-2:** Beam 2 – construction details

The bottom layer of SL72 reinforcement mesh, only provided in the weak side of each beam specimen, was placed directly on top of the steel decking while the top reinforcement layer, also consisting of SL72 mesh, was provided along the entire slab and placed 20mm clear from the top surface of the concrete slab. The spiral enhancing devices and the waveform reinforcement elements used were of the same types as used in the W-Dek® geometry push-out tests described in Chapter 4.2.2 (as shown in Figure 4-8 and Figure 4-10) and were generally provided at the strong side and in the middle region of each beam. The spiral enhancing devices were simply...
clipped over the head of the shear connectors after the studs were welded into position and the waveform reinforcement elements were placed on top of the lap joint in each second rib and additionally tied to the top mesh reinforcement layer.

At the loading points of both beams, formwork for 100mm x 100mm rectangular concrete pockets was provided to allow the load to be directly applied to the steel section (Figure 7-5). This practice had previously been employed (Patrick et al. 1994) in order to prevent any clamping effects of the concrete slab which might be present if the top surface of the concrete slab is loaded directly. As two of the expected failure modes are characterized by horizontal concrete cracks, it was deemed to be crucial to not allow for any clamping effects in these tests either. In Narraine (1984), it was shown that the horizontal shear force distribution is not altered regardless if a single point load is applied to the concrete slab or to the steel section. However, applying the vertical load directly to the steel section was found to create vertical uplift forces in the vicinity of the loading point, hence tensile stresses in the shank of the connectors, whereas in the case of the load applied to the concrete slab, vertical compressive stresses are created in this area (clamping effects). Nevertheless, the uplift effects induced by the loading of the steel section remain
localized and the magnitude of these forces was found to not exceed uplift forces experienced along the shear span of beams where the concrete slab surface is loaded.

**Figure 7-5:** Loading of steel beam through concrete pocket

**Figure 7-6:** Beam 1 – specimen prior pouring

**Figure 7-7:** Beam 2 – specimen prior pouring
The two beam specimens prior to casting are shown in Figure 7-6 and Figure 7-7. Rubber bungs with steel rods which were placed through holes cut into the sheeting directly opposite each pair of studs are also visible in these figures. The rubber bungs were removed before testing and the embedded vertical steel rods were used to provide the measurements of the horizontal slips at the base of each shear connection. The concrete slabs were cast with the steel beams well supported, i.e. propped, along their lengths. Alongside the beams, accompanying push-out test specimens (RIB5-SDM1 to SDM4) and sufficient concrete cylinders to monitor the concrete compressive strength at regular intervals were also poured. All specimens were stored under moist conditions until testing resumed.

### 7.3 Instrumentation of test specimens

The instrumentation of the two composite beam specimens is shown in Figure 7-8. The vertical deflections of the bottom flange of the steel section were monitored at the two potential loading points and at mid-length of each shear span using linear potentiometers (LP1-LP4). The horizontal slips of each shear connection were measured by attaching the tips of the linear potentiometers (LP5-LP16) to the vertical steel rods which had been cast into the concrete slabs next to each shear connection as described above. The linear potentiometers were attached to the top flange of the steel beam section using magnetic bases. Additionally, two linear potentiometers (LP17-LP18) were attached to measure the horizontal end slip of the concrete slab at either side of the beam and five vertical potentiometers (LP19-LP23) were provided to measure the uplift of the concrete slab at selected positions.

Strains were measured at six different locations along each composite beam, two being behind the first shear connection from each support, two behind the second shear connection from each support and two in front of the shear connection closest to the loading point. As it was expected that the stresses in the steel beam in the cross-sections close to the supports would remain in the elastic range, two strain gauges, one being attached to the steel section top flange and one to its bottom flange, were deemed to be sufficient for these locations (see section A-A in Figure 7-8). For the two cross-sections closest to the loading points where yielding of the
steel section was expected, a total of ten strain gauges each were used, eight being attached to the steel section and two to the top surface of the concrete slab (see section B-B in Figure 7-8). The strain gauges attached to the steel section were positioned where the influence of the residual stresses was thought to be relatively small; hence, the top and bottom flange gauges were positioned at 1/6 of the total flange width (24mm) measured from its outside edges and the web gauges were positioned at quarter heights of the steel section (63mm) measured from the outside edge of the top and bottom flanges respectively. Note that the tensile coupons for determining the material properties were also taken from these cross-section locations in accordance with AS/NZS3679.1 (Standards Australia 1996). The gauges on the top surface of the concrete slab were positioned at 300mm transverse centres from the centreline of the specimen.

The readings of the strain gauges attached to the steel section were verified in a bending test of the bare steel beam. The beam was loaded up to 70kN maximum load via one of the designated loading points. The strain gauge measurements for Beam 2 are shown in Figure 7-9. The dotted lines represent the predicted readings for the measured steel beam geometry assuming elastic bending theory. They seem to provide a good estimate for the actual measurements. Note that where more than one gauge per section width was used, the predicted strength approximately equalled the
average of both readings. The predicted bending strains in the particular cross-sections along the beam were also very well reflected by the strain gauges as shown in Figure 7-10 for some of the strain gauge locations in Beam 2. (Note: the average value of the two gauge readings per section width is used as the test value) Therefore, it was decided that no further calibration of the strain gauges was necessary.

**Figure 7-9:** Steel gauge readings at bare steel beam test of Beam 2

In addition, the strains in the shanks of the studs for the shear connectors placed closest to the supports on either side of the beams were measured. The strain gauges were fitted in a 2mm diameter central hole, 20mm below the head of the stud and sealed with special adhesive in the same manner as the stud strain gauges used for the push-out tests (see also Chapter 4.2.2.1).
7.4 Test set-up and procedures

Once the concrete reached its target strength, the specimens were lifted into position with the ends placed on the roller supports. For the Beam 2 specimen which was of an eccentric concrete slab lay-out, a counterweight $G$ was used in order to shift the centre of gravity to the centreline of the steel section (Figure 7-11). The counterweight was attached to a transverse steel section which passed through one of the voids between the steel section and concrete slab to the other side of the steel beam and was welded at its end to a longitudinal steel section which distributed the counterweight over several concrete ribs (in order to keep the tensile stresses induced into the concrete slab low) at a sufficient distance from the shear connections.

The test procedure for each composite beam is summarized in Table 7-2 and illustrated in Figure 7-12. Initially, a load was applied directly to the steel beam top flange through the concrete pocket provided, as shown in Figure 7-5, to the weak (conventionally reinforced) side of the specimen (Test A). The load was cycled ten times between 10kN and 40% of the expected composite beam strength. Further cycles were also undertaken at 60% and 80% of the expected strength. After the last cycle was completed the load was continuously increased until the shear connections

Figure 7-10: Steel gauge readings for various cross-sections at different load levels in bare steel beam test of Beam 2
at the weak side of the beam reached their maximum strengths and experienced initial failure. However, it was not desired to fail the shear connection completely at this stage as composite action was still required in this weak shear span during testing of the stronger side. Hence, Test A was stopped when the load-deflection curve had significantly flattened and most of the load slip-curves of the weak side shear connections had experienced substantial plastic deformations indicating that the composite beam had approached its maximum capacity and shear force redistribution between the individual shear connections had started to appear with the majority of the connections reaching their maximum strength. Special caution was required not to induce too much slip into the shear connections positioned closest to the support as a relatively brittle failure mode with limited slip capacity was expected for the weak side. Therefore, a complete redistribution of the horizontal shear forces between the individual shear connections was not achieved at this stage. As shown in Table 7-2, the initial weak side tests were stopped at end slips of the concrete slabs well past 2mm. From push-out tests, it was observed that the maximum load in similar conventionally reinforced specimen usually occurred at slips of 2mm or less which is another indication that the shear connections close to the support must have approached or exceeded their maximum capacities.

![Figure 7-11: Counterweight for Beam 2 specimen](image)
Table 7-2: Summary of test set-ups and procedures for composite beam tests

<table>
<thead>
<tr>
<th>Test set-up</th>
<th>Beam 1</th>
<th>Beam 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test A</td>
<td>A</td>
<td>B1</td>
</tr>
<tr>
<td>Test A</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td>Test B1</td>
<td>10</td>
<td>4</td>
</tr>
<tr>
<td>Test B2a</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Test B2b</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Test C</td>
<td>2.4</td>
<td>3.8</td>
</tr>
<tr>
<td>Test B2</td>
<td>0.5</td>
<td>2.3</td>
</tr>
</tbody>
</table>

1: Insufficient height of restraint (see Figure 7-13), 2: Insufficient stiffness of clamping device.

Figure 7-12: Test procedures composite beam tests

Once Test A was completed, the load was removed and the composite beam was moved to the position that the load could be applied to the loading point at the strong side of the beam (Test B1). Here, the load was cycled in a similar fashion as in Test A between 10kN and 40%, 60% and 80% of the expected composite beam strength. It was then intended to apply the load continuously until failure or significant slip deformations of the strong shear span of the composite beams were reached as the
shear connections were designed in a manner that this maximum strength could be accommodated by the weak side shear connections. However, during testing of Beam 1, it was observed that once the strong side shear connections close to the support had lost significant amounts of their stiffness, i.e. they started to experience substantial plastic deformations, disproportionate increases in slip for some of the weak side shear connections were measured whereas the slips on the strong side connections only experienced minor changes. Hence, from this point on, the weak side shear connections of the beam rather than the strong side connections were being tested.

![Figure 7-13: Initial horizontal end restraint of Beam 1 – Test B2a: Punch-through failure due to insufficient height](image)

![Figure 7-14: Higher end restraint to limit longitudinal slip of weak side connections in Beam 1 – Test B2b](image)

The test was stopped and it was decided to prevent an increase in horizontal slip at the weak side of the beam by attaching a stiff end restraint to the concrete slab (Test B2). The end restraint initially used for Beam 1 turned out to be of insufficient height as a punch-through type of failure occurred where the concrete surrounding the restraint broke out from the remainder of the slab in Test B2a (Figure 7-13). Consequently, a new end restraint of increased height and width was used (Figure 7-14) where no such failures were experienced, and, in Test B2b of Beam 1, the strong side of the beam could be tested until a sufficiently large end slip of 7.8mm was reached. This slip was well in excess of the 6mm shear connection slip which defines a ductile connection according to Eurocode 4 (CEN 2004). Again, it was
desired to not completely fail the strong side shear connections as a total failure of these connections (if possible) would not allow retesting of the weak side connections where the deformation capacity of the shear connections had not yet been comprehensively investigated.

For the Beam 2 tests, it was decided to initially provide the weak side end restraints for the strong side shear connection test, i.e. Test B1 was waived. However, in the preceding weak side test (Test A), a horizontal rib shearing crack had already developed in one of the concrete ribs (see also Chapter 7.5.3), which, during the course of Test B2 started to propagate across the concrete rib and, additionally, a new horizontal crack formed in the adjacent concrete rib as the cover part of the concrete slab started to delaminate from the concrete ribs. Hence, the weak side shear connections rather than the anticipated strong side connections were being tested towards the end of test B2. Subsequently it was decided to suppress this vertical concrete slab separation by clamping the concrete slab and the steel section of the weak side shear span together. A steel section, which was placed parallel to the steel beam on the top surface of the concrete slab and tied via steel rods to the bottom flange of the steel section, was used as a clamping device (Figure 7-15). Therefore, 16mm diameter holes needed to be drilled into the concrete slab in order to provide a passage for the steel rods. The clamping device initially used was found to be of insufficient stiffness allowing the slab separation to proceed (Test B3a). An increase in the number of steel rods and the application of transverse angles under the steel beam rather than flat steel plates (see Figure 7-15) overcame this problem. The strong side of Beam 2 could then be tested until failure of the concrete slab compression zone occurred at the end of Test B3b (see Chapter 7.5.3).

Once the strong side tests were stopped, the specimens were moved again so that the load could be applied to the weak side loading point. All restraining elements were removed and the testing resumed until complete failure of the weak side shear connections had occurred (Test C).
7.5 Test results and observations

7.5.1 General

The properties of the steel beam section, the steel decking and the headed studs were determined in tensile tests in accordance with AS1391 (Standards Australia 2005). The material properties are summarized in Table 4-17.

<table>
<thead>
<tr>
<th>Material</th>
<th>Yield strength $f_y$ (MPa)</th>
<th>Ultimate strength $f_u$ (MPa)</th>
<th>Elastic Modulus $E$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>250UB31.4 steel section, flanges</td>
<td>334</td>
<td>517</td>
<td>203000</td>
</tr>
<tr>
<td>250UB31.4 steel section, web</td>
<td>351</td>
<td>521</td>
<td>215000</td>
</tr>
<tr>
<td>W-Dek® steel decking, $t=0.75$mm</td>
<td>626</td>
<td>629</td>
<td>200000</td>
</tr>
<tr>
<td>W-Dek® steel decking, $t=1.00$ mm</td>
<td>579</td>
<td>587</td>
<td>184000</td>
</tr>
<tr>
<td>Studs, 127mm high</td>
<td>-</td>
<td>550</td>
<td>-</td>
</tr>
</tbody>
</table>

The self-weight of the composite beams was generally added to the jack load in order to obtain the total load. So was the counterweight in Beam 2. However, as the beam was lifted into position, the effects of the self weight on deflection, shear connection...
slip and strain measurements could not be obtained and were neglected in the following analysis. The effects were estimated to be small.

### 7.5.2 Beam 1 – internal stud pair application

Beam 1 was tested over a period of four days. The average measured concrete compressive strength over that time was 31.9 MPa. The load deflection behaviour of the composite beam is shown in Figure 7-16.

![Figure 7-16: Load-deflection behaviour of Beam 1](image)

Test A was stopped at a load of 252.5kN when the load-deflection curve of the weak side loading point (LP2) had flattened which indicated that the weak side of the composite beam had reached its ultimate bending capacity. A longitudinal concrete crack at the concrete slab surface propagating along the centreline of the slab from the concrete pocket towards the end of the slab was the only visible damage to the concrete beam (see also Figure 7-19). This crack is thought to be sufficiently contained by the transverse top and bottom reinforcement. At the end of Test A, four of the five shear connections had already experienced significant plastic
deformations with slips of about 2mm for the three shear connections closest to the support (Figure 7-17 and Figure 7-18) which, if compared to the accompanying push-out tests, would already indicate that these connections had reached their maximum strength. However, the connection closest to the loading point (LP9) had experienced significantly less slip at this stage and was not fully utilized.

![Graph](image)

**Figure 7-17:** Load-slip behaviour of weak side shear connections and slab end in Tests A & C of Beam 1

When loading on the weak side of the composite beam resumed in Test C, the specimen did not reach its earlier strength (Test A), hence, the shear connection strength was already reduced at that stage. In the further course of the testing, it could be observed that the concrete slab separated from the steel decking along the entire weak side shear span and diagonal cracks formed between the lap joints and the upper corner of the concrete ribs in the ribs where a lap joint was existent (see Figure 7-19). Initially, the composite beam continued to lose strength but was able to eventually regain some of it towards the end of testing which was probably caused by strain hardening effects experienced in the steel section. From Figure 7-17, it can also be seen that while the slab end slip (LP17) continued to increase, the slips experienced by the shear connections (LP5-LP9) hardly changed in Test C. This
Composite beam tests

Clearly indicated that concrete failure surfaces surrounding the shear connections had developed along which the concrete cover slab was able to move longitudinally with little further resistance. The test was eventually stopped at about 19mm end slip when the concrete slab came close to touching the steel angles which were previously welded to the top flange as part of the restraining device used in Test B2.

Figure 7-18: Longitudinal slips of shear connections along the shear spans at various load levels

Figure 7-19: Weak side of Beam 1 after end of test
After testing was finished, the concrete slab was cut transversely at the weak side loading point to separate the two different halves of the concrete beam. The weak side concrete slab could then be easily removed from the remaining beam with the help of a forklift (Figure 7-20). The cone-type concrete failure surfaces surrounding the studs in the sheeting pans characterizing stud pull-out failures then became visible (Figure 7-21).

**Figure 7-20:** Removal of concrete slab from weak side of composite beam after concrete slab was cut transversely at loading point.

**Figure 7-21:** Stud pull-out failures of weak shear span connections in Beam 1

In the strong side tests B1 and B2, the total load could be increased up to a maximum load of 316.6kN. Immediately after this maximum was recorded, the horizontal end
restraint at the weak side of the beam failed (see Figure 7-13) which accounted for the subsequent slight drop in load in Test B2a (Figure 7-16). Once the horizontal restraint was improved and the test restarted (Test B2b), the beam was able to regain loads of a similar magnitude. Tests B2b was continued up to an end slip of close to 8mm where the load-deflection curve of the strong side loading point (LP3) had flattened and total loads of about 314kN were experienced. During testing, a longitudinal concrete crack propagated along the centreline of the concrete slab in a similar fashion as observed for the weak beam side in Test A. Additionally, some small cracks at the transverse edges of the concrete ribs in the vicinity of the loading point developed (Figure 7-23). A slight bulging of the steel decking in the direction of the longitudinal shear force indicating rib punch-through failures were also observed for some of the ribs.

![Graph showing load-slip behaviour](image)

**Figure 7-22:** Load-slip behaviour of strong side shear connections and slab end in Tests B1 & B2 of Beam 1

The slip distribution of the individual connections and the slab end slip in Figure 7-18 and Figure 7-22 show that the horizontal shear forces were reasonably well distributed between four of the five shear connections which all experienced slips in excess of 4mm with the shear connection closest to the support (LP16) even
reporting a slip of more than 6mm at maximum load. The shear connection closest to the loading point (LP12) had a slip of about 1.2 mm indicating that while it must have transferred a significant amount of the longitudinal shear forces, it was not fully utilized during the Tests B1 and B2. In contrast to the weak side, the slips experienced at the end of the slab (LP18) were of similar magnitudes as the slips of the closest shear connection (LP16) and no separation between concrete slab and steel decking was observed during testing. Hence, all shear connections must still have been intact when Test B2b was stopped.

![Strong side of Beam 1 after end of test](image1)

**Figure 7-23:** Strong side of Beam 1 after end of test

![Strong side shear connections after longitudinal cut of concrete slab](image2)

**Figure 7-24:** Strong side shear connections after longitudinal cut of concrete slab

After testing, the concrete slab of the strong side was cut longitudinally just next to the steel section flange (Figure 7-24). It can be seen that while horizontal stud pull-out cracks had developed in most of the concrete ribs, their propagations and successive failures of the shear connections were successfully contained by the
longitudinal bars of the waveform elements which crossed these cracks vertically and effectively ensured continuous load transfer. Diagonal cracks between the base of the shear connector and the top corner of the concrete rib had also developed in most of the connections indicating the formation of a wedge breaking out of the concrete rib, i.e. the onset of rib punch-through failure. In the rib where the spiral enhancing device is visible, the concrete had well penetrated into the device (although it excluded the larger aggregates) and remained virtually undamaged.

### 7.5.3 Beam 2 – stud pair edge beam application

Beam 2 was tested on three consecutive days. The concrete compressive strength over that time averaged 34.0MPa. The load deflection behaviour of the beam test is shown in Figure 7-25.

![Figure 7-25: Load-deflection behaviour of Beam 2](image)

Test A was stopped when the first rib shearing crack started to appear in the concrete rib closest to the loading point at 237.9kN. At that point, the load-deflection curve of the weak side loading point (LP2) had already significantly flattened and the shear
connections closest to the support had experienced significant slips in the order of 1.5mm (Figure 7-26 and Figure 7-27). Hence, the weak shear span was close to reaching its maximum capacity. A longitudinal crack, propagating from the concrete pocket along the centreline of the concrete slab towards the weak end of the slab, similar to the one observed in the previous Beam 1 tests, was the only other crack visible at that stage.

**Figure 7-26:** Load-slip behaviour of the weak side shear connections and slab end in Tests A & C of Beam 2

During the strong side tests (B2 & B3a), the horizontal rib-shearing crack on the weak side continued to propagate across the entire rib and another rib shearing crack also started to appear in the adjacent rib until the test set-up was adjusted to prevent any further separation of the slab (see also Chapter 7.4 and Figure 7-15). Hence, when Test C was performed, the maximum load of the weak shear span remained slightly below its maximum capacity achieved earlier in Test A. During the further course of Test C, rib shearing cracks continued to develop successively in adjacent ribs until the cover slab had completely separated from the concrete ribs along the entire weak side shear span at the longitudinal concrete edge (Figure 7-28). The development of the rib shearing cracks was accompanied by a constant drop in load. The load only started to stabilize when rib shearing cracks in each concrete rib had
occurred. The strength of the composite beam must then have mainly comprised of the bare steel beam strength, which, assuming full plasticity of the critical steel cross-section, would have accounted for about 132kN of the total load of 187kN. At that point, the test was stopped as presumably all the shear connections in the weak side shear span had failed. Similar to Beam 1, the shear connection response was very brittle. Four of the five shear connections had experienced slips of less than 5mm when the test was stopped. The end slip of the slab (LP17) was much larger than the slip of the closest shear connection (LP5) indicating that the cover slab had completely separated from the shear connections and moved longitudinal over the predominantly horizontal failure surface. During testing, a transverse crack between the two last concrete ribs had also appeared at the top surface of the concrete slab (Figure 7-28). This crack is similar to the ‘back-breaking’ cracks observed in push-out specimens (see Chapter 3.1.2.2) and is thought to not influence the shear connection behaviour if sufficient longitudinal top reinforcement is provided, as was the case in this beam test.

![Figure 7-27: Longitudinal slips of shear connections along the shear spans at various load levels](image)
Figure 7-28: Cracking of the concrete slab at weak side of Beam 2 during Test C

After testing, the concrete slab was cut transversely at the weak loading point and the concrete cover slab could be easily removed from the remaining beam (using a forklift). Figure 7-29 shows the rib shearing failure surfaces after the cover slab was removed. The failure surfaces propagated horizontally from the close edges of the concrete slab above the head of the shear connectors and formed a stud pull-out cone at the internal side of the concrete slab. The bottom layer of reinforcement mesh, which was laid on the steel decking ribs, proved to be ineffective in preventing this type of failure as was also observed in the earlier push-out tests.

Figure 7-29: Rib shearing failures at weak side of Beam 2 after concrete slab was cut and the cover part removed
In the strong side tests (Tests B2 & B3), the total load could be increased to 295.3kN when a sudden failure of the concrete compressive zone at the narrow side of the concrete slab appeared (Figure 7-31). As the load-deflection curve of the strong side loading point (LP3) was still inclined at this point (Figure 7-25), it can be assumed that the shear connection had not been loaded to its maximum capacity. However, most of the shear connections had reached slips in the vicinity of 3mm at slab failure and considerable force redistribution between the individual connectors must have occurred (Figure 7-27 and Figure 7-30). The shear connection closest to the loading point (LP12) had experienced 1.2mm slip and can be considered to have transferred a significant amount of force although it was not fully utilized. Other cracks of the concrete slab, which had occurred during the course of these tests, included small vertical and horizontal cracks in the concrete rib closest to the loading point (Figure 7-31) and a transverse ‘back-breaking’ crack at the top surface of the concrete slab similar to the transverse crack in the weak beam side observed in Test C.

![Figure 7-30: Load-slip behaviour of strong side shear connections and slab end in Test B2 & B3 of Beam 2](image-url)
After testing, the strong side of the concrete slab was cut longitudinally next to the steel beam flange on the internal side. All concrete ribs appeared to be sound with the exception of the shear connection closest to the support where a rib punch-through crack was visible (Figure 7-32). This is another indication that the shear connections had not reached their full capacity when slab failure occurred. However, horizontal concrete rib shearing cracks might have existed and, if the slab would have been cut closer to the shear connection or on the other side of the steel section, they might have become visible. In any case, it can be concluded that, at slab failure, the
reinforcing components were effective in suppressing the effects of rib shearing and, in most ribs, the onset of rib punch-through failure.

7.6 Analysis of composite beam tests

7.6.1 Behaviour of stud shear connectors

7.6.1.1 General

The results of the steel beam and concrete slab strain gauge measurements are shown in Figure 7-33 for various load levels of Test A, Beam 2, for the cross-section including the strain gauges SG5-12 and SG25-26. As long as the steel section behaviour remained elastic, the strain gauge measurements resembled the linear strain gradient in the section very well. Hence, for the determination of the steel beam normal forces, only the average top and bottom flange gauge readings needed to be considered. However, when strains exceeding the yield strain of the steel section where experienced, large differences in the measurements of two opposite strain gauges sometimes occurred, as for example in the strain measurements of the bottom flange gauges at maximum load shown in Figure 7-33. Where these differences occurred, the average reading of both strain gauges did not necessary reflect the linear strain gradient of the section. It was therefore decided to extrapolate the average readings of the web gauges to the bottom flange position in order to achieve a linear strain response in these cases. In the steel beam cross-sections closer to the support where only two strain gauges were provided, the strains generally did not reach the yield strains of the material. Consequently, a linear strain gradient between the top and bottom flange strain measurements should provide an accurate prediction of the strain distribution over the height of the steel section. The strain distributions shown in Figure 7-33 also highlight how the distance between the two neutral axes in a composite section increases with increasing loads; hence the shear connection loses horizontal stiffness and the slip at the steel-concrete interface increases.
It is well established that hot-rolled steel sections are subject to significant amounts of residual stress. In order to account for the effect that the residual stresses might have on the onset of yielding in the section, the measured strains were superimposed with residual strains derived from the residual stress distribution suggested for Australian types of hot-rolled steel sections in Bild and Trahair (1989). Therefore, a total of \( n = 120 \) points across the steel section centre-line were discretized as shown in Figure 7-34. Note that the compressive residual strain in the web, \( \varepsilon_{rwc} \), is determined in order to satisfy the stress equilibrium condition

\[
\int_A \sigma_r dA = 0. \tag{7-1}
\]

From the total strain in the discrete elements, their corresponding stresses were determined using the liner-elastic ideal-plastic stress-strain relation given in Figure 7-35 and the measured material properties of Table 7-3. For load cycles, the two cases, elements in the linear-elastic range and elements in the plastic range, needed to
be differentiated so that for any case, elastic unloading and reloading was ensured (see Figure 7-35).

![Diagram](image)

**Figure 7-34:** Assumed residual strain distribution (Bild and Trahair 1989) superimposed on strain measurements

**Figure 7-35:** Assumed stress-strain relation in discrete elements of steel section

The normal force $N$ in the particular steel beam cross-section was found from the stresses in each particular element, $\sigma_i$, and its corresponding area $A_i$ where

$$ N = \sum_{i=1}^{n} \sigma_i A_i. $$

(7-2)

The individual shear connector load $P_s$ was determined as the normal force difference $\Delta N$ between adjacent cross-section measurements in the shear span investigated, distributed evenly among the number of connectors $n_i$ where

$$ P_s = \frac{\Delta N}{n_i}. $$

(7-3)

### 7.6.1.2 Beam 1

The stud force vs. slip behaviours of the various shear connections determined from the slip and strain measurements in accordance with Equations (7-2) and (7-3) are shown in Figure 7-36 for the weak shear span of the composite beam and in Figure
7-37 for the strong shear span respectively. Note that for the shear connections three and four from the slab end, the longitudinal shear forces were evenly distributed among the four studs (two shear connections).

The connections in the weak shear span experienced maximum stud forces between 65kN and 77kN whereas the connections in the strong shear span experienced
maximum forces between 82kN and 128kN. The difference in the stud forces between the two strong side shear connections closest to the end of the concrete slab seems to be uncharacteristically large and might require some adjustment for a more realistic force estimate to account for local end effects. However, the increase in maximum stud force compared to the weak side shear connections would still remain considerable. Prior to Test C, the weak side shear connections had already experienced significant drops in stud force. When Test C was stopped, the forces had further dropped to about 20kN for each stud, hence all connections had effectively failed while all but one shear connection had not even experienced a slip of 6mm. Consequently, the shear connections behaviour in the weak shear span of the composite beam was generally of a brittle nature as none of the connections fulfilled the ductility requirements specified in Eurocode 4 (CEN 2004). In contrast, no significant drops in stud forces were experienced for the strong side shear span of the beam. All shear connections seemed to be able to maintain their capacity with increasing slips, i.e. they displayed a ductile behaviour (Figure 7-37). The slips generally were found to have exceeded 4mm slip in the shear connections investigated with the connection closest to the end of the slab reaching a slip in excess of 6mm, hence, the required deformation capacity to be classified as ductile in accordance with Eurocode 4 (CEN 2004) was experienced for this connection.

**7.6.1.3 Beam 2**

In Figure 7-38 and Figure 7-39, the stud force vs. slip behaviours of the shear connections are shown for the weak and strong shear span of Beam 2. The maximum stud forces were found to be between 55KN and 67 kN for the weak span which is distinctively lower than the maximum forces of the weak span shear connections in the internal beam application (Beam 1). In the strong shear span, the maximum forces experienced were between 80kN and 109KN, which is in a similar range as the maximum stud forces for the strong side internal beam application although slightly lower. The differences in shear connection strengths and behaviours experienced for the weak and strong sides are similar to Beam 1: while the weak side connections experienced a significant drop in stud force which eventually led to failure of the entire shear connection, the strong side connections were able to
maintain the maximum shear connection forces at larger slips until failure of the composite beam occurred. (At that stage the connection closes to the slab end was still experiencing an increase in shear connection strength.) The shear connection behaviours of all but one connection on the weak side were found to be extremely brittle. However, the connection closest to the slab end would have actually fulfilled the ductility requirement of 6mm slip capacity given in Eurocode 4 (CEN 2004). In contrast, no drop in load was observed for the shear connections on the strong side of the beam, i.e. a ductile behaviour was generally displayed, although no statement in regards to the ductility requirement of Eurocode 4 (CEN 2004) can be made as the composite beam failed in slab compression before the required slip capacity of the shear connection was experienced.

![Stud force vs. slip behaviours in weak side shear span of Beam 2](image)

**Figure 7-38:** Stud force vs. slip behaviours in weak side shear span of Beam 2
7.6.2 Comparison with push-out tests

From the four different stud force vs. slip curves in each shear span, an average curve was generated. The curves were obtained by averaging the stud forces for a given value of slip. Each of these average stud force vs. slip curves is compared with the results of push-out tests of similar shear connections in Figure 7-40 to Figure 7-43. As the behaviours of the shear connections after their maximum forces were reached varied strongly across the connections on the weak side shear spans, the falling branches of the average curves are drawn in dotted lines and can only reflect a rough estimate.

The initial stiffness of the shear connections are in very good agreement for the composite beam and push-out tests as are the maximum shear connector forces. However, the shear connections seem to behave slightly stiffer in the beam tests after approximately 60% of the maximum stud forces were reached. The weak side shear connection behaviour, after maximum strength was experienced, appeared to be more brittle for the composite beam shear connection. Hence, the concrete-related failure modes, stud pull-out and rib shearing, had an even more severe impact than was observed in the push-out test. One of the reasons for the more ductile behaviour
in the conventionally reinforced push-out specimens might be the beneficial (and unrealistic) effects of the anchorage of the bottom reinforcement mesh as described in Chapter 4.4.4.

**Figure 7-40:** Comparison of average stud force vs. slip behaviour of weak side of Beam 1 with push-out test result

**Figure 7-41:** Comparison of average stud force vs. slip behaviour of strong side of Beam 1 with push-out test results

**Figure 7-42:** Comparison of average stud force vs. slip behaviour of weak side of Beam 2 with push-out test results

**Figure 7-43:** Comparison of average stud force vs. slip behaviour of strong side of Beam 2 with push-out test results
The results of the tensile forces measurements in the stud shanks for both weak side and strong side connections are compared with the measurements of the accompanying push-out test specimens in Figure 7-44 and Figure 7-45. Again, the measurements in both applications are in very good agreement and it appears that the single-sided push-out test set-up did not induce unrealistically high tensile forces into the shanks of the stud connectors but was rather able to realistically simulate the conditions in the composite beam applications. If at all, the tensile forces in the stud shanks were slightly underestimated in the push-out tests. Hence, the claim made in Chapter 3.1.1.5 that the push-out test set-up should allow for some uplift forces to develop in the shear connectors seems to be justified.

Generally, it can be said that the brittle, concrete-related failure modes experienced in secondary slab push-out tests also occurred in full-scale composite beam applications. The beneficial effects of the waveform reinforcement elements and the stud enhancing devices, suppressing the unwanted brittle failure modes and increasing the shear connection strengths, were confirmed in the beam tests. The
composite beam strength and failure modes experienced could be well predicted from the results of the accompanying push-out tests or the design provisions given in Chapter 6.3.

7.6.3 Influence of stud reinforcing elements on composite beam behaviour

A comparison of the composite beam test results is shown in Table 7-4. The average stud force at maximum load in each shear span was determined using the strain gauge readings of the cross-sections located closest to the loading point and the provisions given in Chapter 7.6.1.1 for all but one shear connection. For the shear connection in the vicinity of the loading point, only the slips were measured. The forces in this particular shear connection were estimated by assuming the appropriate average stud force vs. slip behaviour shown in Figure 7-40 to Figure 7-43.

| Table 7-4: Comparison of composite beam test results |
|---------------------------------|---------------------------------|
|                                  | Beam Test 1 – Internal beam stud pair application | Beam Test 2 – Stud pair edge beam application |
|                                  | Side A (conventionally reinforced) | Side B (waveform element + spiral enhan. device) | Side A (conventionally reinforced) | Side B (waveform element + spiral enhan. device) |
| Max. total load                 | 252.5kN | 316.6kN | 237.9kN | 295.3kN |
| Loading point deflection at max. load | 21.9mm | 67.9mm | 20.5mm | 54.5mm |
| Avg. stud forces in shear span at max. load | 65.8kN | 90.3kN | 55.3kN | 86.3kN |
| Max shear connection slip at max. load\(^\d\) | 2.03mm | 6.00mm | 1.62mm | 3.24mm |
| Min shear connection slip at max. load\(^\d\) | 0.28mm | 1.23mm | 0.22mm | 1.21mm |
| Failure mode                    | Partial shear connection | No failure observed | Partial shear connection | Compressive zone of beam |

\(^\d\): of all shear connections in shear span

The combined use of waveform reinforcement elements and stud enhancing devices increased the capacity of the composite beams investigated by 25% for the internal beam application and 24% for the edge beam application. This increase in strength
can be attributed solely to an increase in the average shear connection strength which amounted to 37% for the internal beam application and to 56% for the edge beam application, as the conventionally reinforced shear spans in both beams experienced failure of the partial shear connection. However, the increase in the average shear connection strength itself is not solely caused by an increase in strength of the individual shear connections but also by their improved slip capacity which ensured a more even force distribution between the individual stud connectors. In the conventionally reinforced shear spans, the shear connections closest to the loading point had experienced very small slips at maximum load (see Table 7-4) which, if the particular average stud force vs. slip curve is assumed to be valid for these connections, would only account for 47%-50% of their capacity. In contrast, the slips of these particular shear connections in the shear spans including the reinforcing elements would have accounted for 89% of their capacity. Even higher shear connections strengths might have been possible for the shear spans where the reinforcing elements were applied as this particular span was not tested to failure in Beam1, and as a composite beam slab compression failure was experienced in Beam 2 before the full shear connection strength could be utilized. The increases in strengths were accompanied by significant increases in deformation capacity of the composite beams, increasing from deflections of span/192 and span/205 at maximum load for the conventionally reinforced sides, which is barely the required serviceability deflection in accordance with AS2327.1 (Standards Australia 2003a) to deflections of span/62 and span/77 for the sides with the reinforcing elements provided. The impact of a concrete edge in the vicinity of the shear connection on the shear connector strength was also markedly different as this edge was found to reduce the average shear connection strength in the conventionally reinforced shear span by 16% but by only 4% for the shear span incorporating the reinforcing elements.

7.7 Summary

Two secondary composite beams, one being an internal secondary beam application and one being a secondary edge beam application, were tested. Both composite beam specimens were divided in two different halves where the shear connections on one
side were conventionally reinforced fulfilling the design provisions of Eurocode 4 (CEN 2004) and the connections on the other side included waveform reinforcing elements and spiral enhancing devices. With the help of separate loading points for each side and an optimized test procedure which, if required, included the application of horizontal and vertical restraints to the untested side of the beam, it was possible to test both sides of the composite beams independently until either failure or sufficiently large deformations had occurred. The use of horizontal linear potentiometers in the vicinity of each shear connection and numerous strain gauge measurements at various composite beam cross-section locations ensured an accurate determination of the stud force vs. slip behaviour for most of the shear connections. In this, the influence that the residual stresses in the hot-rolled steel section had on the onset of yielding was considered.

The test results showed that the concrete-related premature failure modes of stud pull-out, rib shearing and rib punch-through, experienced in earlier push-out tests, also occurred in a full-scale composite beam application. The behaviour of the conventionally reinforced shear connections which experienced stud pullout failures in the internal beam specimen and rib shearing failures in the edge beam specimen appeared to be of an even more brittle nature than in the accompanying push-out tests. The brittle shear connection behaviour resulted in very limited deflection capacity of the composite beams and insufficient force redistribution between the individual stud connections. The use of the reinforcing elements on the other hand overcame the brittle effects of these failure modes by reinforcing the potential failure surfaces. The composite beam load and deflection capacities were significantly increased as the shear connections generally experienced increased stud strengths and deformation capacities which subsequently improved the force redistribution between individual stud connections.

The shear connection strengths and failure modes experienced in the composite beam tests were found to generally compare very well with the results obtained from similar push-out test specimens using the novel single-sided test set-up; even the uplift forces experienced in the shanks of the stud connectors were of similar magnitudes. Therefore, it can be concluded that the results obtained from the other push-out tests described in Chapter 4 would also well reflect the shear connection
behaviour in composite beam applications. Hence, the design approach developed on the basis of these tests, and presented in Chapter 6.3, should provide a good estimate for secondary composite beam shear connections incorporating trapezoidal types of steel decking.
8 DESIGN RESISTANCE OF SECONDARY BEAM STUD CONNECTIONS

8.1 General

In order to apply the new stud strength method for secondary composite beams developed in Chapter 6.3 and found to be in good agreement with the results of composite beam tests described in Chapter 7 to the design provisions given in AS2327.1 (Standards Australia 2003a), a statistical reliability analysis of the new method is required. The current resistance (capacity) factor $\phi$ for shear connectors used in accordance with Table 3.1 of AS2327.1 is 0.85. This factor is re-evaluated as part of the reliability analysis for the stud shearing capacity $P_{\text{solid}}$ by additionally considering the results of more recent solid slab tests as given in Chapter 4.5.1.1. As it is deemed to be favourable to have a uniform capacity factor for all types of shear connection, it was decided to introduce correction factors $k$ for the different types of failure models in the form of:

$$f_{ds} = \min(\phi_{\text{solid}} P_{\text{solid}} \cdot \phi_{\text{RPT, max}} P_{\text{RPT, max}} \cdot \phi_{\text{RPT, min}} P_{\text{RPT, min}} \cdot \phi_{\text{RS/SP}} P_{\text{RS/SP}})$$

$$f_{ds} = \phi_{\text{solid}} \min(P_{\text{solid}}, k_{\text{RPT, max}} P_{\text{RPT, max}} \cdot k_{\text{RPT, min}} P_{\text{RPT, min}} \cdot k_{\text{RS/SP}} P_{\text{RS/SP}}).$$

Note that while the influence of the load-sharing factor $k_n$ given in Clause 8.3.4 of AS2327.1 (Standards Australia 2003a) is not part of this investigation, it is still considered to be valid for the shear connection design regardless of the type of failure mode experienced.
Additionally, similar correction factors are obtained from a reliability analysis of the simplified method given in Chapter 6.3.4 which provides a fast and easy design for the application of the Australian types of trapezoidal steel decking.

### 8.2 Concept for reliability analysis

The limit state design is generally characterized by the design resistance of the member exceeding the design actions for the strength limit state, which is typically a sum of the applied factored load effects, hence

\[ \phi R_n \geq \sum \gamma_i L_i \]  

(8-2)

where \( R_n \) represents the nominal resistance and \( L_i \) the load effects and potential overloads. The resistance factor \( \phi \) reflects the uncertainties associated with the nominal resistance while the load factors \( \gamma_i \) account for the uncertainties of the load effects. The reliability and safety of structural design procedures is commonly described in terms of a reliability (safety) index \( \beta \) which is based on the computed theoretical probability of failure. In general, this can be a complex and lengthy process although good approximations can be obtained with simple methods (Leicester 1985). Assuming that the load effects are statistically independent from the resistance and both reliabilities are lognormally distributed, then using first-order probability the reliability index can be expressed as

\[ \beta = \frac{\ln \left( \frac{R_m}{L_m} \right)}{\sqrt{V_R^2 + V_L^2}} \]  

(8-3)

(Ravindra and Galambos 1978) where \( R_m \) and \( L_m \) are the mean values of the resistance and load effects and \( V_R \) and \( V_L \) the corresponding coefficients of variation.

The relationship between the mean resistance \( R_m \) and its specified nominal resistance \( R_n \) can be written as:

\[ R_m = \frac{R_m}{R_t} \frac{R_t}{R_n} R_n \]  

(8-4)

where the ratio \( R_m/R_t \) takes into account the variability between mean strength as measured in the laboratory specimens (which are assumed to reflect the ‘exact’
Design resistance of secondary beam stud connections

strength) and the theoretical strength function \( R_t \) using test measured dimensions and material properties. The ratio \( R_t/R_n \) takes into account the variability between the variables of the theoretical strength function and its nominal values. The relationship for the corresponding coefficients of variation can be obtained as

\[
V_R = \sqrt{V_\delta^2 + V_{rt}^2}
\]  

(8-5)

where \( V_\delta \) is the coefficient of variation for the ratio of test results to theoretical strength prediction, i.e. the scatter of the test results, and \( V_{rt} \) the coefficient of variation of the variables of the theoretical strength function.

The combined live and dead load parameters, the mean load effect \( L_m \) and the corresponding coefficient of variation \( V_L \), are generally dependent on the ratio of dead load \( G \) to live load \( Q \). The mean load effect \( L_m \) can be expressed as the sum of the mean dead and live loads, \( G_m \) and \( Q_m \), where

\[
L_m = G_m + Q_m.
\]  

(8-6)

The nominal load effects given in limit state designs are typically of the form

\[
L_n = \gamma_G G_n + \gamma_Q Q_n
\]  

(8-7)

where \( \gamma_G \) and \( \gamma_Q \) are determined for the appropriate combination of action investigated at the ultimate limit state. If a factor \( r \) for the ratio of the nominal dead load to the combined total load is introduced, i.e.

\[
r = \frac{G_n}{G_n + Q_n},
\]  

(8-8)

the mean load effect can be written as a function of this ratio:

\[
L_m = L_n \frac{L_m}{L_n} = \frac{\left( r \frac{G_m}{G_n} + (1-r) \frac{Q_m}{Q_n} \right)}{r \gamma_G + (1-r) \gamma_Q} L_n.
\]  

(8-9)

The coefficient of variation can also be expressed as a function of the independent coefficients of variation for dead loads \( V_G \) and live loads \( V_Q \) using the ratio \( r \) of dead to total load given by Equation (8-8):
\[
V_L = \frac{\sqrt{(G_m V_G)^2 + (Q_m V_Q)^2}}{L_m} = \frac{\sqrt{\left( r \frac{G_m}{G_n} V_G \right)^2 + \left( 1 - r \right) \left( \frac{Q_m}{Q_n} V_Q \right)^2}}{r \frac{G_m}{G_n} + \left( 1 - r \right) \frac{Q_m}{Q_n}}.
\]

A detailed investigation into the load combination formulae for Australian Limit State Codes (Pham 1985a) found that the dead loads are typically underestimated and their statistical parameters to be the following:

\[
\frac{G_m}{G_n} = 1.05; \quad V_G = 0.10
\]

For the live loads effects on office floors, the following statistical parameters were given for floor sizes typical for composite beam applications:

\[
\frac{Q_m}{Q_n} = 0.70; \quad V_Q = 0.26
\]

For floor applications, the load factors to be considered for the ultimate stress limit states in accordance with AS/NZS1170.0 (Standards Australia 2002) are:

\[
\gamma_G = 1.35; \quad \gamma_Q = 0 \quad \text{for} \quad r \to 1
\]
\[
\gamma_Q = 1.2; \quad \gamma_Q = 1.5 \quad \text{for} \quad \text{all other } r
\]

### 8.3 Determination of design resistance factors

In the following, the resistance factors \( \phi \) and correction factors \( k \) are determined for the individual failure modes in order to provide an appropriate target reliability index \( \beta_0 \). The relation between resistance factor and target reliability index is obtained by substituting the condition of Equation (8-2) and Equation (8-7) into Equation (8-3) so that

\[
\beta = \frac{\ln \left( \frac{R_m}{R_n} \right)}{\phi L_m / L_n} \geq \beta_0.
\]

No target reliability index is given in the design provisions of AS2327.1 (Standards Australia 2003a). In an earlier reliability study on shear connections in simply
supported beams reliability indices ranging from $\beta=4.5-5.5$ were established (Pham 1984). However, the study was based on an earlier shear connection design method (Pham 1979) which provided significantly lower stud strengths. Other Australian target index reliabilities were set to $\beta_0=3.5$ for steel members (Pham 1985b), $\beta_0=2.5$ for members of cold-formed steel structures (Standards Australia 1998a) and $\beta_0=3.5$ for joints and fasteners in cold formed steel structures (Standards Australia 1998a). Overseas, target indices of $\beta_0=3.8$ for the determination of the various Eurocode models (see Johnson and Huang (1994)) or $\beta_0=3.0$ for composite members in the AISC specifications (see Rambo-Roddenberry (2002)) were used. It was decided to calibrate the design resistance factors against a target reliability index of $\beta_0=3.5$ which represents a probability of failure of $2.3\times10^{-4}$. It should be noted that a different target reliability index would obviously lead to other design capacity factors than the ones presented below.

### 8.3.1 Stud shearing model

The nominal strength shear capacity provision, in accordance with Clause 8.3.2.1 of AS2327.1 (Standards Australia 2003a), for the shear connection strength in a solid slab $P_{\text{solid}}$ is taken as the lesser value of

$$
P_{\text{solid}} = 0.31d_{hs}^2 \sqrt{f_{c} E_{c}} = 0.064d_{hs}^2 f_{c}^{-0.75} \rho_{c}^{0.75},
$$

(8-13)

$$
P_{\text{solid}} = 0.63d_{hs}^2 f_{uc}.
$$

(8-14)

These are investigated as the theoretical strength functions for stud shearing failure where $\rho_c$ represents the density of the concrete.

The comparison of the test results of a total of 163 solid slab push-out test, 87 of which were already presented in Chapter 4.5.1.1 and the remaining 76 were published in Roik et al. (1988), with the theoretical strength functions using either measured material properties or mean values for all variables provided, results in the statistical parameters $P_{m}/P_{t}$ and $V_{\delta}$ as shown in Table 8-1.
Table 8-1: Statistical parameters for evaluation of Equations (8-13) and (8-14)

<table>
<thead>
<tr>
<th>Equation</th>
<th>$n$</th>
<th>$P_{t}/P_{n}$</th>
<th>$V_{k}$</th>
<th>$P_{d}/P_{n}$</th>
<th>$V_{rt}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>(8-13)</td>
<td>87</td>
<td>1.22</td>
<td>0.135</td>
<td>0.99</td>
<td>0.136</td>
</tr>
<tr>
<td>(8-14)</td>
<td>76</td>
<td>1.23</td>
<td>0.102</td>
<td>1.13</td>
<td>0.063</td>
</tr>
</tbody>
</table>

Assuming all variables in the theoretical strength functions to be mutually uncorrelated, the means of the theoretical functions over the nominal functions are determined to be

$$
\frac{P_{t}(8-13)}{P_{n}(8-13)} = \left( \frac{d_{bs,m}}{d_{bs,n}} \right)^2 \left( \frac{f_{cm}}{f'_{c}} \right) 0.75 \rho_{c,m}^{0.75} \rho_{c,n} \quad \text{and} \\
\frac{P_{t}(8-14)}{P_{n}(8-14)} = \left( \frac{d_{bs,m}}{d_{bs,n}} \right)^2 \left( \frac{f_{bc,m}}{f_{bc,n}} \right), \tag{8-15}
$$

while the corresponding coefficients of variation can be determined using the first order Taylor expansion of the theoretical strength function about the mean

$$
V_{rt}^2 = \frac{1}{P_{rt,m}^2} \sum_{i=1}^{m} \left( \frac{\partial P_{t}}{\partial X_i} V_{X_i} X_{i,m} \right)^2 \tag{8-17}
$$

where $X_i$ are the basic variables of the function. Hence, the coefficients of variation amount to

$$
V_{rt,(8-13)} = \sqrt{(2V_{f_c})^2 + (0.75V_{f_c})^2 + (0.75V_{\rho_c})^2} \quad \text{and} \tag{8-18}
$$

$$
V_{rt,(8-14)} = \sqrt{(2V_{d})^2 + (V_{f_d})^2}. \tag{8-19}
$$

The diameter of the stud connectors were generally measured for the tensile tests performed to determine the stud properties as part of the investigations described in Chapter 4 and 5. Hence, a total of $n=37$ measurements were available. The statistical parameters obtained from these measurements were found to be:

$$
\frac{d_{bs,m}}{d_{bs,n}} = 0.99; \quad V_d = 0.008.
$$

These values are in good agreement with the values generally found for steel bolts (see Johnson and Huang (1994)).
From the same stud properties tests, the statistical mean of the ultimate stud strength was determined to be 1.14 if its nominal strength $f_{uc,n}$ is set to 410MPa, the minimum value permitted by AS/NZS1554.2 (Standards Australia 2003b), and the corresponding coefficient of variation was 0.101. However, the studs tested only originated from a few different batches and therefore the individual strengths are not strictly uncorrelated. The statistical values found from the reported stud strengths of the 163 push-out tests investigated are assumed to provide a better estimate. These values were found to be:

$$\frac{f_{uc,m}}{f_{uc,n}} = 1.15; \quad V_{f_c} = 0.06.$$  

The coefficient of variation of the stud strengths seem to agree well with the findings of other researchers (Pham 1979; Roik et al. 1989; van der Sanden 1996) where values between 0.05 and 0.08 were obtained.

The statistical parameters for the concrete strength were chosen in accordance with the assessment undertaken in Pham (1985c) which accounted for the influences of curing procedures, workmanship and the differences between cylinder strength and strengths of concrete cores taken from the finished structures where

$$\frac{f_{cm}}{f'_{c}} = 1.03; \quad V_{f_c} = 0.18.$$  

As part of the push-out and composite beam test series described earlier, a total of 95 concrete density tests were performed on concrete cylinders of normal density concrete. Based on the nominal density of this type of concrete of 2400kg/m$^3$ as specified by AS 3600 (Standards Australia 2001), the following statistical parameters were derived from these tests:

$$\frac{\rho_{cm}}{\rho_{c,n}} = 0.99; \quad V_{\rho_c} = 0.014.$$  

The statistical parameters $P/P_n$ determined from Equations (8-15) and (8-16) and the corresponding coefficients of variation $V_{ri}$ in accordance with Equation (8-18) and (8-19) are given in Table 8-1. It can be seen that the parameters are much more favourable for Equation (8-14), i.e. the provision considering the stud shank strength.
The results of the calculated reliability indices assuming the load factors given by Equation (8-11) are shown for the different ratios of dead load to total combined load $r$ in Figure 8-1. Generally the reliability is much lower for the provision given by Equation (8-13). For the design resistance factor of $\phi_{\text{solid}}=0.85$ as given in AS2327.1 (Standards Australia 2003a), the target reliability index of $\beta_0=3.5$ is not always reached for the typical loading ratios of composite beams ($r \approx 0.4-0.6$). A smaller design resistance factor of $\phi_{\text{solid}}=0.80$, which generally exceeds the target reliability index for Equation (8-13) in this range, is proposed for shear connections experiencing stud shearing failures, e.g. solid slab shear connections. This new design resistance factor would also be in agreement with the stud strength design of BS5950.3 (BSI 1990) and Eurocode 4 (CEN 2004) where a design capacity factor of 1/1.25 (=0.8) is applied.

![Figure 8-1: Variation of reliability indices for the solid slab strength](image)

**8.3.2 Rib punch-through model**

The theoretical strength functions for rib punch-through failures given by Equations (6-32) and (6-33) can both be written in the form:

$$P_{\text{RPT}} = \frac{1}{n_x} \left( P_{\text{conc}} + P_{\text{sh}} \right)$$

(8-20)
Design resistance of secondary beam stud connections

where $P_{\text{conc}}$ is the force component transferred directly into the concrete slab and $P_{\text{sh}}$ the component indirectly transferred into the slab via the steel decking. If the concrete tensile strength $f_t$ is determined by the approximation given in Equation (4-1), i.e.

$$f_t = \frac{1}{2} \sqrt{f'_c},$$

then the concrete force component can be found as

$$P_{\text{conc}} = \begin{cases} \frac{1}{4} h_{\text{eff}} \sqrt{f'_c} \left( s_o + 0.9 b_{0,\text{side}} \right) & \text{for } P_{\text{RPT,max}} \\ \frac{1}{4} \left( h_{\text{eff}} - h_r \right) \sqrt{f'_c} \left( s_o + 0.9 b_{0,\text{side}} \right) & \text{for } P_{\text{RPT,min}} \end{cases}$$

where

$$h_{\text{eff}} = h_e + 1.5 e_t$$

and

$$s_o = s + d_{bs}.$$  (8-23)  

For the purpose of this analysis, only the tensile component of the steel decking is considered as its bending component was found to be negligible for the geometries investigated (see Chapter 6.3.2.6), hence

$$P_{\text{sh}} \approx p_{\text{sh},f} \ell = 4 \frac{T_{sh}}{\sqrt{4 \delta_{sh}^2 + 0.25 \ell^2}} \delta_c.$$  (8-25)  

If it is further assumed that $T_{sh} \approx T_{ysh}$ for the slip deformations considered ($\delta_{\text{sh}}=2\text{mm}/3\text{mm}$), and that the slip deformations remain relatively small compared to the lengths of the sheeting rib $\ell$, hence $4\delta_{sh}^2 + 0.25\ell^2 \approx 0.25\ell^2$, then Equation (8-25) can be simplified to

$$P_{\text{sh}} \approx 8 \frac{f_{ysh} \ell (s_o + b_{h,\text{side}})}{\ell} \delta_c.$$  (8-26)  

It was decided to divide the analysis into two groups, one being for the configuration of single studs in a concrete rib and the other being for stud pairs. It would have been even more preferable to subdivide these groups further into conventionally reinforced shear connections, connections incorporating the stud enhancing devices, connections incorporating the waveform reinforcement element and connections incorporating a combination of both these restraining elements. However, the
number of available test data for some of these groups would have been too small to justify a statistical analysis. If more test data becomes available in the future, it would be recommended to undertake a renewed analysis for each of these individual groups.

The statistical parameters $P_m/P_t$ and $V_δ$, determined from the comparison of the theoretical strength functions with the push-out tests experiencing rib punch-through behaviour (as shown in Figure 6-17 and Figure 6-18), are given in Table 8-2 for the various groups.

**Table 8-2:** Statistical parameters of shear connections experiencing rib punch-through failure

<table>
<thead>
<tr>
<th>Strength function</th>
<th>Studs in pan</th>
<th>$n$</th>
<th>$P_m/P_t$</th>
<th>$V_δ$</th>
<th>$P_t/P_n$</th>
<th>$V_ν$</th>
<th>$k_{RPT}^I$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$P_{RPT,max}$</td>
<td>Singles</td>
<td>15</td>
<td>1.08</td>
<td>0.141</td>
<td>1.02-1.09</td>
<td>0.141-0.230</td>
<td>0.70</td>
</tr>
<tr>
<td></td>
<td>Pairs</td>
<td>22</td>
<td>1.02</td>
<td>0.132</td>
<td>1.22</td>
<td>0.121-0.213</td>
<td>0.85</td>
</tr>
<tr>
<td>$P_{RPT,min}$</td>
<td>Singles</td>
<td>12</td>
<td>1.20</td>
<td>0.147</td>
<td>1.07-1.15</td>
<td>0.145-0.221</td>
<td>0.85</td>
</tr>
<tr>
<td></td>
<td>Pairs</td>
<td>7</td>
<td>1.20</td>
<td>0.239</td>
<td>1.28-1.32</td>
<td>0.123-0.206</td>
<td>0.85</td>
</tr>
</tbody>
</table>

$^I$: correction factor to be used in combination with resistance capacity factor of $\phi=0.8$

The mean of the theoretical strength function is determined as

$$\frac{P_{RPT,t}}{P_{RPT,n}} = \left(\frac{P_{conc,m}}{P_{sh,m}}\right)^{r_p} + \left(\frac{P_{sh,m}}{P_{sh,n}}\right) (1 - r_p)$$  \hspace{1cm} (8-27)

where

$$r_p = \frac{P_{conc,n}}{P_{RPT,n}}$$  \hspace{1cm} (8-28)

$$P_{conc,m} = \left(\frac{h_{eff,m}}{h_{eff,n}}\right) f_c \left(\frac{f_{cm}}{f_c}\right)^{0.5} \left(\frac{s_{o,m} + 0.9b_{0,side,m}}{s_{o,n} + 0.9b_{0,side,n}}\right)$$  \hspace{1cm} for $P_{RPT,max}$

and

$$P_{conc,n} = \left(\frac{h_{eff,m} - h_{r,m}}{h_{eff,n} - h_{r,n}}\right) \left(\frac{f_{cm}}{f_c}\right)^{0.5} \left(\frac{s_{o,m} + 0.9b_{0,side,m}}{s_{o,n} + 0.9b_{0,side,n}}\right)$$  \hspace{1cm} for $P_{RPT,min}$

$$P_{sh,m} \approx \left(\frac{f_{sh,m}}{f_{sh,n}}\right) \left(\frac{t_m}{t_n}\right) \left(\frac{\ell_m}{\ell_n}\right) \left(\frac{s_{o,m} + b_{b,side,m}}{s_{o,n} + b_{b,side,n}}\right)$$  \hspace{1cm} (8-30)
The corresponding coefficient of variation determined in accordance with Equation (8-17) follows as

\[
V_{RPT} = \sqrt{\left( \frac{P_{conc, m}}{P_{conc, n}} V_{p_{conc}} \right)^2 + \left( \frac{1 - r_p}{P_{sh, m}} \frac{P_{sh, n}}{V_{p_{sh}}(1 - r_p)} \right)^2}
\]

where

\[
V_{p_{conc}} = \sqrt{\left( V_{f_{eff}} \right)^2 + \left( \frac{1}{2} V_{f_c} \right)^2 + \left( \frac{s_o}{s_o + 0.9 b_{0,side}} V_{s_s} \right)^2} + \left( \frac{0.9 b_{0,side}}{s_o + 0.9 b_{0,side}} V_{b_{0,side}} \right)^2}
\]

for \( P_{RPT,\text{max}} \)

\[
V_{p_{sh}} \approx \sqrt{\left( V_{f_{sh}} \right)^2 + \left( V_{l} \right)^2 + \left( - V_{l} \right)^2 + \left( \frac{s_o}{s_o + b_{h,side}} V_{s_s} \right)^2} + \left( \frac{b_{h,side}}{s_o + b_{h,side}} V_{b_{h,side}} \right)^2}
\]

for \( P_{RPT,\text{min}} \) (8-32)

The means and coefficients of variation of the individual variables were generally determined from measurements on test specimens or material property tests and are summarized in Table 8-3. The parameters of the strength function given in Table 8-2 account for shear connections of 19mm stud diameter, stud enhancing devices of 76mm diameter, steel decking heights of \( 50 \text{mm} \leq h_r \leq 80 \text{mm} \), stud positions in the sheeting pan being in the range of \( 30 \text{mm} \leq e_b \leq 110 \text{mm} \) and \( 60 \text{mm} \leq e_t \leq 140 \text{mm} \) and, where applicable, transverse stud spacings between \( 80 \text{mm} \leq s_x \leq 120 \text{mm} \). In the specimens investigated, it was found that the proportion of the loads directly transferred into the concrete slabs to the total shear capacity was in the range of \( 0.60 \leq r_p \leq 0.85 \) for the maximum rib punch-through strength \( P_{RPT,\text{max}} \) and in the range of \( 0.30 \leq k_p \leq 0.60 \) for the minimum strength \( P_{RPT,\text{min}} \). These limits are also considered in the determination of the strength function parameters.
Table 8-3: Statistical parameters for individual variables

<table>
<thead>
<tr>
<th>Description</th>
<th>Variable(s)</th>
<th>Number of measurements / tests</th>
<th>Mean</th>
<th>Coefficient of variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sheet thickness</td>
<td>$t$</td>
<td>14</td>
<td>1.13</td>
<td>0.060</td>
</tr>
<tr>
<td>Sheet strength</td>
<td>$f_{sh}$</td>
<td>14</td>
<td>1.15</td>
<td>0.050</td>
</tr>
<tr>
<td>Sheet height</td>
<td>$h_r, t$</td>
<td>19</td>
<td>1.02</td>
<td>0.030</td>
</tr>
<tr>
<td>Sheet pan width</td>
<td>$b_{rt}$</td>
<td>19</td>
<td>1.00</td>
<td>0.010</td>
</tr>
<tr>
<td>Stud position in pan</td>
<td>$e_r (~h_{eff})$</td>
<td>48</td>
<td>1.06</td>
<td>0.070</td>
</tr>
<tr>
<td>Stud spacing (transv.)</td>
<td>$s_o (~ s_o \text{ for stud pairs})$</td>
<td>24</td>
<td>0.99</td>
<td>0.060</td>
</tr>
<tr>
<td>Stud diameter</td>
<td>$d_{bo}$ ($=s_o \text{ for single studs}$)</td>
<td>37</td>
<td>0.99</td>
<td>0.008</td>
</tr>
<tr>
<td>Failed concrete wedge width</td>
<td>$n_c=1$</td>
<td>$b_{0\text{,side}}, b_{b\text{,side}}, b_{t\text{,side}}$</td>
<td>21</td>
<td>0.92</td>
</tr>
<tr>
<td></td>
<td>$n_c=2$</td>
<td></td>
<td>20</td>
<td>1.14</td>
</tr>
</tbody>
</table>

The reliability indices for the different groups, based on a design resistance factor of $\phi = \phi_{\text{solid}} = 0.8$ and the appropriate correction factor $k_{\text{RPT}}$ given in Table 8-2, are shown in Figure 8-2 for the maximum rib punch-through capacity $P_{\text{RPT,max}}$ and in Figure 8-3 for the minimum capacity $P_{\text{RPT,min}}$. The upper and lower limits reflect the influence that the range of parameters has on the determination of the reliability indices.

\begin{align*}
r &= G_n/(G_n + Q_n) \\
\beta &= r^{0.5} \left( \frac{1}{(1 - r)^{0.5}} \right)
\end{align*}

**Figure 8-2:** Variation of reliability indices for maximum rib punch-through strength, $P_{\text{RPT,max}}$
The applications of the chosen design resistance factor in combination with the corresponding correction factor provide safety levels similar to solid slab applications. The correction factors were generally chosen so that the safety indices of the lower limits exceeded the target reliability index in the composite beam load application range of \( r = 0.4-0.6 \). Note that the strength for single stud applications required a lower correction factor in order to provide a satisfactory agreement with the target reliability. The main reason for the larger strength reduction of the single stud application can be found in the prediction method of the concrete failure wedge width which seems to slightly overestimate the width for single stud applications while, on the other hand, the width for stud pair application is underestimated (Table 8-3). In any case, the scatter for the width of the concrete wedge was found to be large, hence the large coefficient of variation for this variable which led to the more unfavourable statistical parameters when compared to stud shearing failures. The differences in the reliability indices between lower and upper limits for the parameters for the various applications investigated are generally fairly large.
8.3.3 Rib shearing and stud pull-out model

The model for rib shearing and stud pull-out failures is given in Chapter 6.3.3. Substituting Equation (8-21) into Equation (6-34) provides the theoretical strength function:

\[
P_{\text{RS}/\text{SP}} = \frac{1}{12} \frac{k_c k_{\sigma} b_{\text{eff}}}{n_c} b_t^2 \sqrt{f'_c}
\]  

(8-34)

where

\[
b_{\text{eff}} = \begin{cases} 
0.9(s_o + b_{t,\text{side}}) & \text{for } P_{RS} \\
 s_o + b_{t,\text{side}} & \text{for } P_{SP} 
\end{cases}
\]  

(8-35)

for specimens of sufficient concrete flange width.

Again, the analysis could only be divided into two groups, shear connections with a single stud connector, and shear connections consisting of stud pairs. Further divisions into shear connections experiencing rib shearing or stud pull-out failures and connections including cross-wires in the concrete ribs would have been desirable but were not possible due to the limited amount of test data available. The statistical parameters for the evaluation of a total of 38 push-out test specimens experiencing these types of failures (as shown in Figure 6-22) are given in Table 8-4.

**Table 8-4:** Statistical parameters of shear connections experiencing rib shearing or stud pull-out failures

<table>
<thead>
<tr>
<th>Strength function</th>
<th>Studs in pan</th>
<th>(n)</th>
<th>(P_m/P_t)</th>
<th>(V_b)</th>
<th>(P_f/P_n)</th>
<th>(V_{rt})</th>
<th>(k_{RS/SP})</th>
</tr>
</thead>
<tbody>
<tr>
<td>(P_{RS/SP})</td>
<td>Singles</td>
<td>9</td>
<td>1.08</td>
<td>0.136</td>
<td>0.92-0.93</td>
<td>0.214-0.269</td>
<td>0.60</td>
</tr>
<tr>
<td></td>
<td>Pairs</td>
<td>19</td>
<td>1.04</td>
<td>0.129</td>
<td>1.06-1.11</td>
<td>0.187-0.253</td>
<td>0.70</td>
</tr>
</tbody>
</table>

\(\phi\): correction factor to be used in combination with resistance capacity factor of \(\phi=0.8\)

The mean and coefficient of variation for the strength function can be derived to

\[
P_{\text{RS}/\text{SP},l} = \left( \frac{s_{o,m} + b_{e,\text{side},m}}{s_o + b_{t,\text{side},m}} \right) \left( \frac{b_{r,m}}{b_{t,m}} \right)^2 \left( \frac{f_{\text{cm}}}{f'_c} \right)^{0.5}
\]  

(8-36)

and

\[
V_{P_{\text{RS}/\text{SP},l}} = \sqrt{\left( \frac{s_o}{s_o + b_{t,\text{side}}} V_{s_o} \right)^2 + \left( \frac{b_{t,\text{side}}}{s_o + b_{t,\text{side}}} V_{b_{t,\text{side}}} \right)^2 + \left( \frac{2V_{b_o}}{b_o} \right)^2 + \left( \frac{1}{2} V_{f_t} \right)^2}
\]  

(8-37)
with the individual parameters given in Table 8-3. As for the rib punch-through reliability analysis, the strength function parameters of Table 8-4 account for shear connections of 19mm stud diameter, stud enhancing devices of 76mm diameter, stud positions in the sheeting pan being in the range of $60 \leq e_t \leq 140$ mm and, where applicable, transverse stud spacings between $80 \leq s_x \leq 120$ mm.

The reliability indices determined using the appropriate correction factor $k_{RS/SP}$ of Table 8-4 in combination with a resistance factor of $\phi=0.8$ are shown in Figure 8-4. Again, the value of the correction factor required is smaller for single stud applications due to the lower mean and higher coefficient of variation of the strength function. The correction factors are found to be generally lower than those determined for rib punch-through failures as the influence of the concrete wedge width to the overall strength is comparably bigger. However, the concrete wedge width is only an estimate used to approximate an effective width (see Equation (8-35)). It is questionable if the effective width would actually display statistical parameters of a similar magnitude. More favourable parameters for the effective width would significantly reduce the correction factor. However, as no such statistical data was obtained, the correction factors given in Table 8-4 can be employed as a lower bound.

![Figure 8-4: Variation of reliability indices for rib shearing / stud pull-out strength](image)

**Figure 8-4:** Variation of reliability indices for rib shearing / stud pull-out strength.
8.3.4 Simplified design for Australian trapezoidal steel decking geometries

The simplified method given by Equation (6-43) can also be applied to the design of stud shear connections in accordance with AS2327.1 (Standards Australia 2003a), i.e.

\[ P_{\text{simplify}} = f_s = k_t P_{\text{solid}}, \quad (8-38) \]

if the reduction factors \( k_{t,m} \) as given by Table 6-6 are calibrated against the target reliability index and the solid slab design resistance factor \( \phi = \phi_{\text{solid}} \) is applied. As the simplified resistance function \( P_{\text{simplify, t}} \) was derived from a parametric study of the general resistance functions \( P_{\text{general, t}} \), the statistical parameters are obtained in the same fashion as

\[ R_m = \frac{P_m}{P_{\text{general, t}}} \frac{P_{\text{general, t}}}{P_{\text{simplify, t}}} \frac{P_{\text{simplify, t}}}{P_{simplify, n}} \quad \text{and} \quad (8-39) \]

\[ V_R = \sqrt{\left( V_{\delta} \right)^2 + \left( V_{rt, \text{general/simplify}} \right)^2 + \left( V_{rt, \text{simplify}} \right)^2}. \quad (8-40) \]

where the mean \( P_{\text{simplify, t}}/P_{\text{simplify, n}} \) and the corresponding coefficient of variation \( V_{rt, \text{simplify}} \) of the simplified strength function are similar to the ones obtained for the stud shearing failures (solid slab applications) as given in Table 8-1. The parameters found for Equation (8-13) were applied to concrete compressive strengths of \( f'_c < 32 \text{MPa} \) and the parameters for Equation (8-14) to concrete compressive strength of \( f'_c \geq 32 \text{MPa} \).

A statistical analysis of the push-out tests with the general strength functions was already performed in Chapter 6.2.3 and the results for its mean \( P_m/P_{\text{general, t}} \) and coefficient of variation \( V_\delta \) are given in respect to the shear connection lay-out in Table 6-2. The statistical parameters for the ratio of general to simplified strength function \( P_{\text{general, t}}/P_{\text{simplify, t}} \) and \( V_{rt, \text{general/simplify}} \) are obtained from the parametric study performed as part of the derivation of the simplified method for each individual group. Consequently, the limits for the application of the simplified method, as stated in Chapter 6.3.4, should be strictly adhered to. Where the reduction factor \( k_{t,m} \) of Table
6-6 was found not to provide the target reliability index, the reduction factor was corrected accordingly.

The reduction factors for the determination of the nominal stud capacity in accordance with Equation (8-38) are given in Table 8-5. These factors can also be directly applied to the nominal solid slab strengths given in Table 8.1 of AS2327.1 (Standards Australia 2003a) to determine the capacity of connections incorporating the two current types of Australian trapezoidal decking if the capacity factor is set to $\phi = 0.8$ and the conditions relating to the application of the simplified method, in particular regarding the stud placements and shear connector height, are satisfied (see Chapter 6.3.4).

### Table 8-5: Reduction factor $k_t$ for secondary composite beam applications for the determination of the nominal stud capacity in accordance with Equation (8-38)

<table>
<thead>
<tr>
<th>Geometry</th>
<th>Number of studs $n_s$</th>
<th>Concrete compressive strength $f'_c$</th>
<th>$&lt; 32\text{MPa}$</th>
<th>$\geq 32\text{MPa}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>$\text{CR}^1$</td>
<td>$\text{WR}^1$</td>
</tr>
<tr>
<td>KF70®</td>
<td>1</td>
<td>N/A</td>
<td>0.80</td>
<td>0.85</td>
</tr>
<tr>
<td>W-Dek®</td>
<td>1</td>
<td>N/A</td>
<td>0.70</td>
<td>0.85</td>
</tr>
<tr>
<td>KF70®</td>
<td>2</td>
<td>N/A</td>
<td>0.50</td>
<td>0.60</td>
</tr>
<tr>
<td>W-Dek®</td>
<td>2</td>
<td>N/A</td>
<td>0.45</td>
<td>0.55</td>
</tr>
</tbody>
</table>

$^1$: CR: conventional reinforced specimens, $^2$: ED: stud enhancing device

$^3$: WR: waveform reinforcement element

$^4$: only applicable for sheeting thickness of $t \leq 0.75\text{mm}$, otherwise N/A

$^5$: only applicable for central positioned studs with a height of $h_c \geq 150\text{mm}$, otherwise N/A

### 8.4 Summary

A reliability analysis was performed on the new method developed for secondary beam stud connections in Chapter 6.3. The analysis was based on a target reliability index of $\beta_0 = 3.5$. It was found that the stud shearing model, also used to predict the solid slab strength in accordance with AS2327.1 (Standards Australia 2003a), required a design resistance factor of 0.80. It is therefore suggested to reduce the capacity factor for shear connections for the strength limit state as given in Table 3.1.
of AS2327.1 (Standards Australia 2003a), from 0.85 to 0.80. Based on this general design resistance factor, correction factors for the strength models of the other failure modes, $k_{RPT,max}$, $k_{RPT,min}$ and $k_{RS/SP}$, were determined to obtain the target level of reliability for these premature failure modes. As these failure modes are generally characterized by cracking of the concrete slab in the vicinity of the shear connection, the coefficients of variation representing the scatter of the strength function were found to be significantly larger compared to the stud shearing provisions. Hence, the nominal strength function needed to be significantly reduced to ensure a similar level of safety in which the correction factors for rib shearing and stud pullout failures were generally lower than those for rib punch-through failure.

The reduction factors $k_i$ for the fast and simplified determination of the allowable shear connection strength for the existing Australian types of trapezoidal steel decking given in Table 6-6 were also calibrated against the target reliability index using the stud shearing design resistance factor of $\phi=0.80$. However, tight limits for the application of these reduction factors apply, and their application should strictly be restricted to the specified types of shear connections.
9 CONCLUSIONS

The behaviour of the shear connection in steel-concrete composite beams is strongly affected by the failure mode experienced. The application of trapezoidal types of steel decking with wide-open ribs can create numerous premature concrete-related failure modes which reduce the strength and deformation capacity of the shear connection significantly and violate the requirement of a sufficiently ductile shear connection to be used in plastic composite beam design. In conventionally reinforced shear connections, these failure modes are characterized by concrete cracks developing along unreinforced failure surfaces. Measures to either delay the onset of cracking or to reinforce potential failure surfaces and ensure continuous load transfer after cracking occurs are effective ways to overcome the brittle effects of these failure modes.

In secondary composite beam applications where the steel decking runs transverse to the steel section, the premature failure modes identified were rib punch-through, rib splitting, rib shearing and stud pull-out.

Rib punch-through and rib splitting failures are localized failures of the concrete rib predominately induced by the vicinity of a free concrete edge in the direction of the longitudinal shear which leave the base of the shear connector unsupported. Their onset was found to be influenced by the position of the shear connector in the concrete rib, the geometry of the concrete rib, the bearing stresses experienced and the resistance provided by the concrete surrounding the shear connection. After cracking, the steel decking which encloses the free concrete edge provides additional longitudinal resistance due to bending and tensile force effects. It was shown that the
application of round steel ring or spiral wire stud performance-enhancing devices placed around the base of stud connectors reduces the bearing stresses while, at the same time, increases the stiffness of the shear connection base which enables a shear force transfer directly into the higher regions of the concrete slab. A bottom layer of transverse rib reinforcement placed at a low position in the concrete rib was also shown to be advantageous in accommodating transverse splitting forces and strengthen the bearing zone directly in front of the shear connection. Note in solid slab applications, such transverse bottom reinforcement is mandatory in current design codes and standards.

Rib shearing and stud pull-out failures, on the other hand, are characterized by the concrete ribs delaminating from the concrete cover slab for which the failure surface propagates between the upper corners of the concrete ribs while locally passing over the heads of the stud shear connectors. Depending on the embedment length of the studs into the concrete cover slab and the width of the concrete rib, the failure surface can be near-horizontal which makes the application of conventional horizontal mesh reinforcement ineffective. Other factors influencing the onset of these failures were found to be the concrete tensile strength, the number of shear connectors per group and the height of the concrete rib. For shear connections with relatively narrow concrete flanges, rib shearing failures were experienced which, as the concrete flange becomes wider, converted into stud pull-out failures. The strain measurements in the stud connectors and the concrete ribs indicated that, contrary to current understanding, these types of failure are not induced by large tensile forces in the shank of the studs but rather by bending effects in the concrete ribs in a manner similar to concrete corbel applications. The application of vertical reinforcement bars, as part of a waveform reinforcement element crossing the anticipated failure surface at the rear of the concrete rib over an effective width, was found to suppress the effects of the two failure modes. However, tests demonstrated that transverse bottom reinforcement bars are required as integral parts of the waveform elements in order to sufficiently anchor the longitudinal bars and to ensure ductile shear connection behaviour.

The evaluation of numerous design approaches that take into account the reduced strength of the shear connection in secondary beam applications did not yield
satisfactory agreements with the results of push-out tests performed on specimens incorporating the Australian types of trapezoidal steel decking. Furthermore, none of these approaches distinguished between brittle and sufficient ductile shear connection behaviour. Hence, a new design method has been proposed that differentiates between the various failure modes and specifies the suitable reinforcing measures to ensure ductile shear connection behaviour. One of the features of this new proposal is to allow for the occurrence of some of the brittle failure modes as long as a minimum capacity can be guaranteed at any given slip up to the required slip capacity. The new method was found to provide increased reliability and much reduced scatter for the strength prediction of stud connectors. However, as a large number of parameters need to be considered, the practicality of this method is somewhat restricted. Based on this method, simple strength reduction factors have been given for the most common applications of existing types of trapezoidal Australian decking geometries. Both the general and the simplified method were calibrated to provide a similar level of safety as the current AS2327.1 (Standards Australia 2003a) design provisions for stud connectors. Based on the reliability analysis performed, it is recommended to reduce the resistance factor $\phi$ for stud shear connectors designed to AS2327.1 (Standards Australia 2003a) from 0.85 to 0.80 to obtain an appropriate level of safety.

In primary beam applications where the steel decking runs parallel to the steel section, a concrete haunch is created. This concrete haunch can experience longitudinal splitting failure along the line of shear connectors or haunch shearing failure between the upper edges of the haunch if it remains unreinforced. In solid slab applications, transverse bottom and vertical stirrup reinforcement needs to be provided in concrete haunches. It is not hard to understand why similar provisions have not been adopted for haunches created in composite slab application in any of the overseas standards as the steel decking surely does not provide any resistance against near-horizontal haunch shearing failure, for example. In the tests performed, it was shown that both failure modes display similar characteristics to longitudinal shear failures with the main influence factors appearing to be the concrete tensile strength, the haunch width and the longitudinal shear connector spacing as an indication of the applied shear force per unit length. In further agreement with longitudinal shear failure provisions, the application of reinforcement crossing the
anticipated longitudinal failure surface, i.e. transverse base reinforcement for longitudinal splitting failures and vertical haunch reinforcement for haunch shearing failures, was found to be able to contain these types of failures and the full shear connection strength to be utilized (for single stud arrangements). Based on the limited amount of test data available, some preliminary reinforcing provisions were given for haunches in composite slab applications where, in any case, transverse haunch reinforcement is required, while the provision of vertical haunch reinforcement is subject to the longitudinal shear strength of the concrete slab across the projected failure surface. However, further investigations, in particular regarding reinforcement requirements for stud pair applications in concrete haunches, are strongly recommended.

It was shown that the test method can have a significant impact on the shear connection behaviour, in particular where concrete-related failure modes are experienced. Measures which suppress the development of the potential failure surfaces or which activate frictional forces, such as lateral restraints or the application of normal forces, should be avoided unless it can be proven that comparable effects permanently exist in the composite beam application investigated. A versatile single-sided horizontal push-out test rig was developed which provides an economic alternative to vertical double-sided push-out test set-ups and allows for a large range of applications to be tested. The results of the small-scale push-out tests performed on this new test rig compare well with the shear connection behaviour and failure modes observed in full-scale composite beam applications. The claim that the push-out tests only provide a conservative estimate of the shear connection capacity as it introduces additional tensile forces into the shear connection cannot be justified for the secondary beam tests undertaken where tensile forces of a similar magnitude were measured in the shanks of stud connectors.

Even though a total of 91 push-out tests have been conducted as part of this investigation and additionally many more tests found from the literature have been considered, the number of geometric and material parameters is such that a comprehensive study of all parameters was impossible. However, it is considered that a general understanding of the various failure modes which occur in shear connections in composite beams incorporating trapezoidal steel decking and their
potential weakening effects has been obtained. The application of various reinforcing measures to subsequently improve unsatisfactory shear connection behaviour and regain sufficient ductility in the connection, which in many cases resulted in stud strengths and deformation capacities similar to solid slab applications, has also been successfully demonstrated. However, the following aspects would require further investigation:

- The studies undertaken focused on 19mm diameter stud connectors which are the most common application. However, AS2327.1 (Standards Australia 2003a) additionally allows for studs of 15.9mm diameter which would experience similar behaviours when used in trapezoidal steel decking shear connections and, hence, these connectors could also benefit from the application of the reinforcing elements.
- Stud performance-enhancing devices of 76mm diameter (and 40mm height in the case of the steel ring) were the only geometry to be tested. Investigation into different geometries of the device could probably improve its benefits and efficiency.
- All waveform reinforcing elements investigated were generally of a similar lay-out consisting of four longitudinal wires spaced at 150mm transverse centres and providing 20mm clear cover to the edges of the concrete ribs. Investigations into different lay-outs could also contribute to the optimization of this element.
- More investigations into the behaviour of the haunch reinforcement elements are highly recommended, in particular for stud pair applications. The anchorage of the ladder and handle-bar reinforcement components and the validity of the proposed reinforcement requirements are other aspects which require further clarification.
- Full-scale primary beam tests including the haunch reinforcement components are advisable to validate its beneficial effects and to rule out any appearance of brittle cover slab failures which were observed in some of the small-scale push-out tests.
- The design provision which takes into account the beneficial effects of transverse reinforcement bars provided in the concrete ribs where rib punch-through and rib splitting failures are experienced is very crude (see
Chapter 6.3.2.5) due to the limited amount of test data available. A more detailed investigation into this behaviour would be recommended.

- Finite element studies of the various failure modes experienced could help to refine the secondary beam design provisions, in particular in regards to the stress distributions experienced across the concrete ribs at the time of failure. They could further be beneficial in the optimization process of the reinforcing elements. However, it should be noted that an accurate three dimensional concrete fracture and post-cracking analysis would be required to satisfactory model the propagation of the failure surfaces.
References


References


DIN (1981). "Richtlinien für die Bemessung und Ausführung von Stahlverbundträgern." Richtlinien für Stahlverbundträger, Deutsches Institut für Normung e.V.


References


References


Standards Australia (2005). "Concrete structures (Revision of AS 3600-1)." DR05252 Draft for Public Comment, Standards Australia.


Viest, I. M. (1956). "Investigation of stud shear connectors for composite concrete and steel T-beams." Journal of the American Concrete Institute, 27(8), 875-891.


