Experimental and Analytical Study of an Innovative Ultra Long-Spanning Hybrid Steel Deck

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A Thesis Submitted For The Fulfilment Of Requirements For
The Degree of Doctor of Philosophy

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The only true wisdom is in knowing you know nothing.

(Sokrates, 470-399 BC)
PREFACE

This thesis is submitted to the University of Western Sydney, Australia, for the degree of Doctor of Philosophy. The work described in this thesis was carried out by the candidate during the years 2003-2006 in the School of Engineering at the University of Western Sydney. The candidate was mainly supervised by Emeritus Professor Russell Q. Bridge, after Dr. Mark Patrick resigned from the University of Western Sydney in 2004 to pursue a consulting career in a commercial company.

Although this thesis was carried out on a commercially sensitive proprietary product, which in turn restricted the publication of detailed information and test data of the product, seven supporting papers have been written.


In accordance with the By-Laws of the University of Western Sydney governing the requirements for the degree of Doctor of Philosophy, the candidate submits that the work presented in this thesis is, to the best of his knowledge and belief, original except as acknowledged in the text. The author hereby declares that this material has not been submitted, either in full or in part, for a degree at this or any other institution.

Mathias Gläsle
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NOTATIONS

LATIN LETTERS

\(a\) distance from centre of TOX connector to corner of web
\(A_{eq}\) equivalent cross-sectional area
\(A_{hb}\) net cross-sectional area
\(A_{sb}\) gross cross-sectional area
\(A_{BP}\) base panel cross-sectional area
\(A_{TP}\) top plate cross-sectional area
\(A_{0d}\) percentage elongation at failure over a 80 mm gauge length
\(b\) width of the joint
width of tensile coupon
distance from centre of TOX connector to edge of flange
width of corrugation
width of compressive element
\(b_e\) effective width of compressive element
\(b_c\) distance from web-flange intersection to centre of connector
\(b_1, b_2\) empirical correction terms for sensors with digital output
\(c\) principal distance
\(C\) compressive force
\(C_{TP}\) resultant compressive force in top plate
\(d\) punch diameter
diameter of reinforcement bar
depth of corrugation
\(d_p\) internal clinch diameter on punch side
\(E\) Young’s modulus of elasticity
\(E_0\) initial elastic modulus
\(E_1, E_2, E_3\) modulus of elasticity in principal direction 1, 2 and 3
\(f\) factor of joint orientation (includes type of manufacturing)
stress
\(f_{cr}\) critical buckling stress
\(f_{BP}\) stress at centroid of base panel
\(f_{Euler}\) Euler stress
\(f_{el}\) elastic buckling stress
\(f_u\) ultimate tensile strength of the material
stress at ultimate load
\(f_{u,BP}\) tensile strength of base panel material
\(f_{uh}\) ultimate stress based on net cross-sectional area
\(f_{um}\) average ultimate stress capacity of actual corresponding solid specimens
\(f_{us}\) ultimate stress based on gross cross-sectional area
\(f_y\) yield strength of the material
yield stress
\(F_{Rd}\) design shear resistance of round press clinches
\(F_{TOX}\) TOX shear capacity
\(G\) shear modulus
\( G_{12}, G_{13}, G_{23} \) shear modulus for plane 12, 13 and 23

\( i \) inside diameter of joint on punch side

\( i_a \) inside diameter (version A)

\( i_b \) inside diameter (version B)

\( I_{\text{eff}} \) localized flexural stiffness over effective length \( L_{\text{eff}} \)

\( k_{\text{eff}} \) continuous flexural stiffness over connector spacing \( s \)

\( k_{\text{red}} \) reduced inside diameter

\( l \) second moment of area

\( I_{\text{P}} \) second moment of area of the panel

\( h_t \) equivalent bending stiffness of connector-member

\( h_{\text{TP}} \) second moment of area of the top plate

\( l_w \) equivalent bending stiffness of web

\( l_z \) second moment of area about z-axis

\( i \) (index) loading epoch \( i \)

\( k \) connector stiffness

\( L \) length of connector-member (\( = y_w - y_c \))

\( l \) span

\( L_{\text{eff}} \) effective length of upper web-flange

\( L_s \) length between panel support and critical cross-section

\( M \) moment capacity (at critical section)

\( M_0 \) moment capacity of panel exhibiting complete shear connection

\( M_u \) ultimate shear load

\( M_{\text{eff}} \) effective length of upper web-flange

\( M_{\text{init}} \) applied ultimate moment at midspan

\( n \) number of connectors

\( n \) exponent for interaction relationship

\( n_c \) number of connections required for complete shear connection

\( N \) number of connections between panel support and critical cross-section

\( N_{\text{max}} \) maximum tension value (mean for 2 TOX)

\( o \) outside diameter of joint on die side

\( o \) (index) initial loading epoch

\( P \) shear load

\( P_{\text{init}} \) load at initiation of fracture

\( P_u \) ultimate shear load

\( P_{\text{fracture}} \) ultimate load

\( P_{\text{1}}, P_{\text{2}} \) decentring distortion parameters

\( q \) empirical design factor expressing strength of basic camera station configuration
\( r \) coupon transition radius  
radius of corner of section  
\( s \) connector spacing  
spacing of support members  
\( S \) image scale (1:S)  
\( s_c \) critical connector spacing  
\( s_{\text{holes}} \) web opening spacing (centre to centre)  
\( s_{\text{TOX}} \) TOX connector spacing  
\( s_u \) slip at ultimate load  
\( t \) sheet thickness  
\( T \) tensile force  
\( t_b \) base metal thickness  
base metal plate thickness  
\( h_{\text{BP}} \) base panel thickness  
\( T_{\text{BP}} \) resultant tensile force in base panel  
\( t_c \) coated thickness  
\( t_d \) die side material thickness  
\( t_{\text{eff}} \) effective web thickness  
\( h_{\text{hor}} \) horizontal projection of upper sheet thickness at the bottom  
\( t_{\text{min}} \) smallest neck thickness on punch side  
\( t_p \) punch side material thickness  
\( t_{\text{TP}} \) top plate thickness  
\( t_w \) full web thickness  
\( t_{\text{WB}} \) web thickness  
\( t_x \) member thickness in X-direction  
\( V \) shear value (for 2 TOX)  
\( V_{\text{max}} \) maximum shear value (mean for 2 TOX)  
\( w \) punch width of press joining tool  
\( x \) remaining bottom thickness (X-dimension)  
position along span  
\( x_p \) principal point offsets in x-direction  
\( y \) distance from neutral axis  
\( y_c \) distance between centroids of upper and lower beam  
\( y_{\text{BP}} \) distance between centroids of base panel and hybrid section  
\( y_p \) principal point offsets in y-direction  
\( y_s \) distance between neutral axis and outer fibre  
\( y_{\text{TP}} \) distance between centroids of top plate and hybrid section  
\( y_w \) web height  
\( z \) vertical point position
GREEK LETTERS

\(\alpha\)  
loading angle 
factor reflecting the influence of initial imperfection and residual stresses

\(\alpha_c\)  
clinching bearing coefficient

\(\beta\)  
degree of shear connection

\(\beta_t\)  
tensile strength

\(\delta\)  
vertical displacement 
deformation

\(d_0\)  
midspan deflection at ultimate moment

\(\Delta_0\)  
separation at ultimate load

\(e\)  
strain

\(e_{0.2}\)  
strain at 0.2 % proof stress

\(e_{200}\)  
percent elongation over a 200 mm gauge length

\(\phi\)  
factor to account for changes in both thickness and steel strength 
capacity reduction factor

\(\gamma\)  
partial material factor

\(\gamma_{M2}\)  
partial factor for calculating design resistance of mechanical fasteners

\(\lambda\)  
load factor against buckling 
buckling half-wavelength

\(\lambda_{Av}\)  
average buckling half-wavelength

\(\lambda_{Av,\text{Edge}}\)  
buckling half-wavelength for the edge plate element

\(\lambda_{Av,\text{Middle}}\)  
buckling half-wavelength for the middle plate element

\(\lambda_{FSM}\)  
buckling half-wavelength (finite strip method)

\(\lambda_{Glob}\)  
buckling half-wavelength of global buckling mode

\(\lambda_{Loc}\)  
buckling half-wavelength of local buckling mode

\(\lambda_{Max}\)  
maximum buckling half-wavelength

\(\nu\)  
Poisson’s ratio

\(\sigma\)  
stress

\(\sigma_i\)  
measurement error at loading epoch \(i\)

\(\sigma_{m}\)  
measurement error

\(\sigma_0\)  
measurement error at initial loading epoch

\(\sigma_{0.2}\)  
0.2 % proof stress

\(\tau_u\)  
ultimate shear strength in the direction of the force

SYMBOLS, MATRICES, VECTORS

\(\{D\}\)  
eigenvector

\([G]\)  
stability matrix

\([K]\)  
stiffness matrix

\(\emptyset\)  
rod diameter
ABSTRACT

A new, ultra long-spanning, combined steel formwork and reinforcement system has been developed in conjunction with the University of Western Sydney (Australia) and is designed to span in excess of eight metres without propping. The hybrid steel deck comprises a cellular module incorporating a top plate with multiple longitudinal stiffeners that is mechanically connected to a base panel through vertically-corrugated webs. All of these elements can have different thicknesses and steel grades. To establish the mechanical connections between the components of the main decking panel, round press-joints (TOX®) are used. The webs can have pre-punched holes to partially or completely fill the closed cellular section with concrete and to allow reinforcing bars and/or prestressing cables be passed transversely. The modules, which can vary in their overall height, can be precambered to control deflection under the weight of the wet concrete.

Small-scale tests have been carried out to understand the basic behaviour (shear and combined shear-tension) of these unusual mechanical connectors, as well as their effect on steel performance, in particular on high-strength steel (G550). In a series of preliminary positive bending tests, a range of principal failure modes could be identified, of which the longitudinal shear failure of the TOX® connectors in the base panel (BPSH) and the buckling of the top plate under compression (TPBU) were most common. These two failure modes were further investigated in an extensive series of flexural tests in which the material combinations and the spacing of the connectors was varied.

For the BPSH failure, measurements of longitudinal slip showed a fundamentally different distribution compared to the conventional theory used for composite steel-concrete beams. A simple computational model was developed to predict the deformation performance, whereas the ultimate moment capacity could be well predicted by using a partial shear connection model in conjunction with elastic theory.

For the TPBU failure, digital close-range photogrammetry was employed to measure the top plate deformations. Notwithstanding the different geometries and material grades used for the top plates, increasing the connector spacing did not significantly reduce the plate strength. Specimens with a low-strength top plate material were able to reach compressive stresses close to yield of the material. Simple design models, based on the results of the finite strip method and an elastic member buckling analysis, have been established to predict the observed buckling behaviour of the top plate. The predicted behaviours were found to be sensitive to the modelling of the hybrid steel deck.
Chapter 1. Introduction

1.1 GENERAL
Aesthetics, engineering and economic parameters are the main factors for finding optimal constructional systems and the long history of structural design reflects this continuous progress. In regard to structures, the search for optimality can mostly be found in improvements of the form and materials used in the construction. For instance, the synergy between structural components made of different materials can result in very efficient systems by utilizing the advantages of each part, with steel-concrete composite construction as probably the most important representative. The principle of this construction method was laid at the end of the 19th century in Europe, and composite bridges and buildings appeared in the United States of America (U.S.) as early as 1894 (Moore, 1988; Viest, 1992). In the years following World War II, the cost of labour in the U.S. increased sharply and the use of wood forms for cast-in-place concrete in building structures became less attractive. This was the time when steel-deck floors, acting both as a permanent form for the wet concrete fill and working platform during the construction stage, came to the fore. In using cellular steel floors (the first cellular steel floor, called “keystone beam”, was laid in the early 1930’s in Pittsburgh, U.S. by H.H. ROBERTSON Co.), the weight of the concrete could be significantly reduced which resulted in big savings in structural steel and foundation costs. Another advantage of cellular floor systems was that the electrical and communication wiring could be channelled through the ducting of the floor cells which made this type of floor most suitable for office buildings (Dallaire, 1971).

In the early use of steel-deck floors, the steel decking was the only structural element whereas the concrete acted primarily as the filling material. This changed in 1950 when the first composite steel-deck-reinforced concrete floor system (known as “Cofar” and produced by GRANCO STEEL PRODUCTS Co.) appeared on the U.S. market (Schuster, 1974). The corrugated steel sheet was less steel-intensive and acted compositely with the hardened concrete as a result of metal wires (T-wires)
Chapter 1. Introduction

welded transverse to its top surface. In the early 1960’s, an even more economical deck was developed by INLAND-RYERSON Co., with small depressions embossing the surface of a cellular deck to provide interlock with the concrete. This innovative cellular deck was later used as the floor system for the 110 stories tall Sears building in Chicago (Dallaire, 1971). Today, composite slabs incorporating cold-formed profiled steel sheeting are the preferred type of floor in steel-frame buildings in most developed countries. This is largely due to the advantages steel decking provides, which makes it economically competitive to alternative flooring systems, such as precast slabs or conventional reinforced concrete slabs. Among the many advantages steel decking has (Fisher and Buettner, 1979; Patrick, 1998; Porter and Ekberg, 1976; Wright et al., 1987), the elimination or significant reduction of the positive moment reinforcement and formwork for concrete casting are two of the most important ones.

Unpropped composite steel decks have become increasingly popular to promote rapid construction but deck spans have, for many years, typically only been in the range of two to three metres. This has controlled the spacing of secondary steel beams, and to a large degree the economics of the composite steel-frame system. In increasing the span lengths by about two times, significant cost savings can be expected (Widjaja and Easterling, 2000) and therefore research in the area of long-span slabs has been carried out (Hillman and Murray, 1994; Lawson and Mullet, 1993; Ramsden and Segerlind, 1986; Stark, 1986). Commercially, very deep trapezoidal decks with wide steel ribs, possibly over 200 mm high, have been developed for use in slim-floor construction in composite steel-frame buildings. Using this approach, the depth of the decking is hidden within the depth of the steel beams by either hanging the decking off the top flange (e.g. German ADDITIV DECK) or supporting the decking on the bottom flange (e.g. British COMFLOR SD225 deck). Unpropped spans in the vicinity of five to six metres have been achieved, sometimes involving a two-stage pour to reduce the weight of the wet concrete. However, being roll-formed from flat steel sheets limits their structural efficiency due to a constant material thickness around their perimeter, and increasing sheeting thickness is the normal way to span further with a given profile. Also, it is difficult to control vertical deflection under the weight of wet concrete and significant ponding can occur. This is wasteful of concrete and has a compounding effect by increasing the overall mass of the structure and the size of supporting elements. Although concrete savings may result due to the voids created by the wide open trapezoidal ribs, they create a variety of problems including weakening the shear connection formed with the steel beam when shear connectors are used, reducing the fire resistance and acoustic performance of the finished slab, and limiting two-way action.

A different approach to increase spanning capability was taken by the TRUSSDEK system which was first released into the Australian construction market in 1999 and achieved spans in excess of 5.5 m in the formwork stage without propping (Patrick, 2002). In its initial form (as shown on the left in Figure 1-1), it combined steel decking with conventional steel reinforcing trusses that allowed distribution of the material in an advantageous way by concentrating steel in the extreme tensile and compressive regions. Although being successfully used at the time, the pursuit for an even more efficient, longer-spanning system began and resulted in an entirely new version (TRUSSDEK II)
where the reinforcing trusses have been replaced by roll-formed elements (Patrick et al., 2004). This hybrid steel deck includes a number of innovative features and is designed to span in excess of eight metres with a flat level soffit at the completion of pouring. The deck consists of a number of complex cold-formed steel components which are mechanically connected to each other by round press-joints to produce a closed cellular section, as seen on the right of Figure 1-1. The main decking panel, which can take on different heights, compromises three basic components, viz.: a stiffened base panel with overlapping joints; vertically corrugated webs with and without pre-punched holes; and a longitudinally stiffened top plate. All elements can have different thicknesses, steel grades and coatings depending on the particular application of the hybrid steel deck. There are also pre-fitted end-diaphragms and intermediate formers to allow hollow and filled zones of concrete. A detailed description of the TRUSSDEK II system is given in Chapter 2.3.

![Figure 1-1. TRUSSDEK I (left) and TRUSSDEK II (right) system (Photos: OneSteel Reinforcing)](image)

### 1.2 Motivation and Scope of Research

The research carried out in this thesis is entirely focused on the new, innovative hybrid steel deck (TRUSSDEK II) which was developed in conjunction with the University of Western Sydney, Australia. The key element for long-spanning, unpropped steel decks used in steel-concrete composite constructions is the ability to carry the wet concrete loads and to control the deflections before the concrete hardens. Therefore, the performance of a steel deck in the bare metal state can be more critical than its performance in the actual composite state. For that reason, the research in this thesis is strictly limited to the performance of the hybrid steel deck in the bare metal state.

At the beginning of this research project, only very limited tests on prototypes have been carried out to show the feasibility of the new hybrid steel deck. Since the closed cellular section is formed by a number of elements connected to each other using intermittent, discrete mechanical connectors, it is expected that the load carrying behaviour is rather different to conventional steel decks and further research is required. Furthermore, the elements can have different geometries, thicknesses and material properties thus increasing the need to have a good understanding of the components of the deck and how they interact.

To verify the capability and reliability of this new and complex steel deck, experimental testing was essential to obtain an understanding of the system, in particular when being used as a product for practical applications in commercial projects. This also involved a basic experimental investigation of
the unusual mechanical connectors (round press-joints) and their effect on the behaviour of the hybrid steel deck.

The final scope of research was developed during the course of preliminary testing where potential failure modes of the hybrid steel deck could be identified in a series of single span tests. The tests gave a better understanding of these modes which involved buckling of elements in compression and/or shear, yielding/fracture of elements in tension and the failure of the mechanical connectors due to the thin-walled nature of the structure. The effect of end-diaphragms on the overall performance of the deck was also examined. Further investigations into the most dominant failure modes were made and let to the development of appropriate engineering models.

1.3 OUTLINE OF WORK

The following is a brief outline of the major topics covered in this thesis:

- Chapter 2 describes the development and innovative approach of the ultra long-spanning hybrid steel deck. It also contains a comprehensive description of the system and of the mechanical connectors (round press-joints) used to form the cellular section.

- Chapter 3 contains the results of small-scale tests carried out to understand the shear behaviour of these unusual mechanical connectors, as well as their effect on steel performance, in particular with high-strength steel (G550).

- Chapter 4 presents a series of preliminary positive bending tests of the hybrid steel deck. On the basis of these tests, decisions were made regarding the significant aspects of the behaviour that would be more closely investigated in the research of this thesis. It also contains a numerical investigation into the effects that an open end-diaphragm would have on the local and global buckling of the hybrid steel deck.

- Chapter 5 contains the results of an extensive series of small-scale tests to study the behaviour of the connectors used in the hybrid steel deck and loaded in combined shear and tension. Based on these results, interactive relationships between shear and tension have been proposed.

- Chapter 6 presents the results of a series of flexural tests carried out to study the longitudinal shear failure of the mechanical connectors in the base panel of the hybrid steel deck by varying the connector spacing and material thicknesses. It also contains a simple computational model that was developed to model the slip behaviour of the panels and to predict the deformation performance of the hybrid steel deck. Various partial shear connection models were used to predict the ultimate moment capacity of the tested panels.

- Chapter 7 describes the non-contact measurement technique (Photogrammetry) employed to determine the initial imperfections of the top plate and its out-of-plane distortions during flexural testing. In using off-the-shelf digital cameras and a suitable photogrammetric software package, even a non-specialist user of the photogrammetric technique could achieve highly accurate measurements up to 0.02 mm in this application.
• Chapter 8 presents the results of a series of flexural tests carried out to study the buckling behaviour of the longitudinally stiffened and discretely connected top plate of the hybrid steel deck. Different top plate geometries/thicknesses were investigated to establish the effect of varying the connector spacings. The influence of different material grades and web thicknesses were studied. Simple design models were developed, based on the results of the finite strip method and an elastic member buckling analysis. These models predict the buckling behaviour of the top plate observed in the experimental programme.

• Chapter 9 contains a summary of the findings from this research and the conclusions reached. In addition, the necessity for future research is discussed.

• Appendix A contains a detailed description and the results of the tensile coupon tests

• Appendix B contains all test results for the TOX shear tests (Chapter 3.1)

• Appendix C contains all test results for the effect of TOX joints on steel performance (Chapter 3.2)

• Appendix D contains all test results for the combined shear-tension TOX tests (Chapter 5)

• Appendix E contains all test results for base panel shear tests (Chapter 6)

• Appendix F contains photogrammetric data of the top plate buckling tests (Chapter 8)

• Appendix G contains the traced outlines of the top plates tested in Chapter 8.

• Appendix H contains all tests results for top plate buckling tests (Chapter 8)

NOTE: All product brand names and company names used throughout the thesis are indicated in capital letters and may be subject to copyright or registration.
2.1 GENERAL

Composite slabs incorporating profiled steel sheeting have a long history in Australia (Bridge and Patrick, 1996). In 1965, John Lysaght reviewed various profile shapes for the development of an Australian product. The new profile, known as BONDEK, had narrow dovetail ribs with wide flat pans and was based on the British HOLORIB profile (Lysaght, 1965). With no embossments on the ribs, bond between the galvanized deck and the concrete was achieved solely by chemical adhesion of the cement paste to the zinc coating, limiting maximum unpropped span and overall slab thickness. In the early 1990’s, BONDEK II was developed which had embossments along the top of the ribs to enhance mechanical and frictional resistance between the steel decking and hardened concrete (Patrick, 1998). The steel deck was therefore no longer dependent on adhesion bond for its anchorage and could span further than its predecessor.

In 1999, after nearly five years of research and development work in conjunction with the University of Western Sydney, an innovative long-spanning steel decking system, called TRUSSDEK, was first released into the Australian construction market (Patrick, 2002). It allowed spans in excess of 5.5 m in the formwork stage without propping and was a lightweight, permanent, combined steel formwork and reinforcement system, with reinforcing trusses welded to the pans of high-tensile galvanized profiled steel sheeting (BONDEK II, but others were viable too) which formed the bottom chord of the trusses, as can be seen in Figure 2-1. By welding the trusses, which came in a variety of heights, to the profiled steel sheeting, the mechanical interlock between the decking and the hardened concrete could be even further enhanced, and the steel decking could be treated as fully-effective longitudinal reinforcement in the final slab or beam (Patrick and Grey, 2003).
Chapter 2. Development of an ultra long-spanning steel deck

The high-tensile (G550) steel decking had a large tensile capacity that had the potential to displace large quantities of conventional reinforcement, noting that the top chord bars were also used structurally in the final slab. The narrow, re-entrant open (dovetail) steel ribs stiffened the base sheet, and longitudinal stiffeners in the pans precisely located the trusses. Being able to vary the number of the prefabricated trusses per panel (two or three) and the size of the truss top chord had the advantage of concentrating steel in the extreme compressive regions. Even additional top bars could be welded to the underside of the top chord bar to strengthen the panels. Triangulated wire webs transferred shear forces and laterally restrained the top chord to increase its buckling capacity.

Another concept realized with the TRUSSDEK system was the development of special voids for achieving lightweight floor construction and also controlling the vertical deflection of the deck suspended between adjacent trusses. Non-structural types of voids could replace individual trusses to reduce the volume of concrete and therefore the weight of the floor, provided that the remaining trusses were strong enough to carry the required loads. Deep void formers resembling the steel ribs of deep trapezoidal decks could be clipped between the dovetail ribs, or very shallow pan plates could transfer the weight of the wet concrete to adjacent trusses without interfering with longitudinal or transverse reinforcing bars or prestressing cables that had to be placed in the slab.

2.2 FURTHER DEVELOPMENTS

Although the TRUSSDEK system was being successfully used at the time, the pursuit of an even more efficient, longer-spanning system began about two years later. Improved methods for manufacturing the product were also investigated. In particular, alternative ways of connecting the trusses to the steel decking were studied which would allow the use of sheeting with a pre-painted soffit. Other forms of the trusses were considered for this purpose. For example, welding small angles to the bottom of the web chords was trialled, which were then glued to the steel deck, as can be seen on the left of Figure 2-2. The application of adhesives to connect structural steel members is possible and efficient (Pasternak et al., 2004) and tests carried out at the University of Western Sydney showed that the structural adhesive could be used economically to achieve similar strengths to the welded connection in even the thickest base metal thickness.
Chapter 2. Development of an ultra long-spanning steel deck

It then followed that a void could be introduced in the same process by gluing the folded trapezoidal section (as shown on the right in Figure 2-2) to the decking. This also allowed the effective length of the web chords to be reduced by welding them to the sides of the void, thus allowing smaller diameter chords to be used. Nevertheless, the product still involved a mix of roll-formed decking and reinforcing steel, and several connection processes, so did not progress beyond the prototype stage.

Further consideration of previous prototypes and desirable features led to an entirely new version of the TRUSSDEK system. The main top reinforcing bar was replaced by a steel plate welded to the top flange of a larger trapezoidal rib. The BONDEK II sheeting was replaced with a narrow deck roll-formed again from high-tensile galvanized steel, but with very shallow edge ribs (see Figure 2-3).

The top steel plate formed the main compression element and could be made out of inexpensive uncoated steel of a lower grade than the other elements (encased in concrete in the composite stage). Testing showed that it was essential to fit a diaphragm at each panel end (as shown in Figure 2-3) where the support reaction was concentrated. The web was stiffened to increase its vertical shear strength. This was achieved by pressing vertical corrugations into the web, which would also increase the mechanical interlock developed between the web and the concrete in the final slab. To accommodate the vertical corrugations, the webs would have to be produced in two individual parts.

Initially, the webs were glued to the steel deck but an alternative means was sought to speed up the production process. In the development of a thin-walled structure, the selection of the connection type becomes very important, not only for structural considerations but also because the connection...
contributes, to a substantial extent, to the overall costs. Apart from structural requirements, such as strength, stiffness and deformation capacity, non-structural requirements have to be considered for the selection of the connection type as well. These could involve aesthetics, watertightness, durability (e.g. chemical aggressiveness of the environment, possible galvanic corrosion, etc.) and obviously economic aspects (total number of connectors needed, skills/tools required, costs of installation of the connection, etc.) which can vary for different connection types (Toma, 2003). Round press-joints were considered to meet the above requirements for the new TRUSSDEK version.

2.3 HYBRID STEEL DECKING SYSTEM

The present version of the TRUSSDEK system (TRUSSDEK II) is designed to span in excess of eight metres without propping and to provide control of vertical deflections in the formwork stage which occur mainly due to the weight of wet concrete. The ultra long-spanning hybrid steel decking system is based on a modular approach, consisting of a number of complex cold-formed elements which take on efficient shapes, thicknesses, steel grades and coatings to give the system great versatility compared to other composite steel decks. A number of innovative features make the system adaptable to almost any type of building construction ranging from conventional, shallow one-way composite slabs in steel frame buildings to deep two-way post-tensioned slabs in concrete-frame buildings. The system is also suitable for civil works, such as mining projects, where very deep concrete slabs (up to 500 mm) have to be supported without propping. Examples of different types of construction for which the hybrid steel decking system has been used are given elsewhere (Komselis et al., 2005b).

![Figure 2-4. Main hybrid steel deck panel with a section height of 140 mm](image)

The main hybrid steel deck panel (see Figure 2-4) comprises three basic components, viz.: a stiffened base panel with small over-lapping, dovetail joints; vertically-corrugated webs with or without pre-punched holes; and a longitudinal stiffened top plate. The components are mechanically connected together using round press-joints after they have been roll-formed to produce a torsionally stiff, closed cellular section. The height of the panel varies depending on the application with a minimum of 90 mm to a maximum of 260 mm, and an additional four intermediate sizes (110, 140, 160 and 210 mm) between these limits. Even with the varying heights the components are made from the same roll-forming equipment. Diaphragms are fitted at the ends of the panels where they fall on
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The main hybrid steel deck panels are laid parallel and interconnected for maximum spanning capability, however for efficiency different types of infill panels may be fitted between the main hybrid steel deck panels.

The cellular section can form a void in the concrete of a one-way composite slab, or it can have holes pre-punched in the webs allowing it to be filled with concrete. The concrete infill may occur locally at any critical region (using internal void formers to block concrete flow) or completely along its length to make a solid composite slab. Major savings in the volume of concrete used, and thus significant savings in the overall weight, can be achieved by voiding the cellular section. Partial fillings with concrete might be beneficial, for example, at the ends of panels where there are welded-stud shear connectors in a composite steel-frame construction. The solid concrete slab is then advantageous in avoiding rib punch-through and other types of secondary failure of the shear connection. Two-way composite slab action can be achieved by passing transverse reinforcing bars or prestressing cables through the pre-punched holes of the web and filling the voids as necessary. Prestressing cables can also be fitted longitudinally between the main hybrid decking panels to achieve very long spans of the finished composite slabs by reducing the immediate vertical deflection if propping is used.

A very important feature of the ultra long-spanning hybrid steel deck is the ability to control the vertical deflections in the formwork stage if used unpropped (a feature which has not been possible with conventional composite steel decks). This is accomplished by upwardly pre-cambering the main hybrid steel deck panel in the longitudinal direction to a designed height during manufacture. Using the appropriate amount of camber, good visual appearance and general alignment of the slab soffit can be achieved which leads to considerable savings of ponded concrete. To achieve a truly level soffit, the base panel (as well as the infill panels) may also be transversely cambered in the factory.

2.3.1 Base panel

The base panel is specially designed to suit the unique demands of the system and is made of thin cold-formed high-tensile galvanized steel (G550) that provides significant tensile or compressive capacity acting as longitudinal reinforcement in the composite slab. It is manufactured using the roll-forming process, as shown on the left in Figure 2-5. The longitudinal stiffeners in the pan serve a number of purposes, viz: (i) they reduce the degree of waviness of the flat surfaces (known as oil canning) which can otherwise be the source of large imperfections and amplify local buckles as well as being a problem aesthetically; (ii) they improve the compressive capacity of the panel in negative moment regions; (iii) their outer edges are used to precisely locate the webs (180 mm apart at the base); and (iv) they make the panel compact in negative or positive bending.

The narrow, re-entrant open steel lapping ribs are only 20 mm high so, unlike profiles with deep ribs, they do not impact on the behaviour of the slab in the transverse direction. The passage of reinforcing bars or prestressing cables in the transverse direction to provide two-way action in a solid slab is then possible. The base panel has a constant nominal width of 250 mm for all section heights and is available in different base metal thicknesses up to 1.0 mm. Prominent upwardly-oriented
dimple embossments regularly rolled into the top of the larger overlapping rib provide extra longitudinal shear resistance in the composite slab which supplements the resistance provided by the mechanical connections along the pan. Tests carried out on the base panel (other components were also tested) show that the shear resistance in composite action can be very high, being about 370 kPa at 6 mm slip (Patrick et al., 2005). The lap joint is specially designed to form a leak-proof joint and has a winged extension on the overlapping rib that makes assemblage of the panels easy.

The shallow design of the base panel, together with the strong mechanical interlock or resistance of the steel ribs in the hardened concrete, even allows the use of the base panel on its own to form a unique composite steel deck (ONEDEK) in its own right.

2.3.2 Corrugated web and holing

A pair of vertically-corrugated webs is mechanically connected to the base panel and the top plate through the upper and lower web flanges to form the cellular section. A lower grade galvanized steel (typically G350) was chosen for economics, and also because the steel has high formability to assist in the fabrication of the corrugations. The vertical corrugations with a pitch of 35 mm are pressed into the webs to increase their shear capacity as well as the mechanical interlock developed between the web and the hardened concrete. The webs (produced in a nominal base metal thickness of either 0.7 or 0.9 mm) are inclined at a fixed angle of 74 degrees to the horizontal irrespective of the panel height.

Standard holes 90 mm long and 40 mm deep (deeper holes are readily available) may be punched in the webs immediately prior to the roll-forming operation (as shown on the right in Figure 2-5), either at a regular spacing (of typically 200 mm centres) or locally situated at chosen locations to meet specific requirements. The holes are normally punched centrally within the depth of the web, but can also be positioned at other heights. To improve the flow of concrete into the voids, the holes are normally staggered on opposite sides, but may be directly opposite each other when reinforcing bars or cables are to be passed through the sections. Just below the upper web flange, small air-vent holes are punched into the webs to prevent air from being trapped in the void.

![Figure 2-5. Roll-forming operation of base panel (left) and corrugated web (right)](image)

2.3.3 Top plate

Finite strip analysis (THIN-WALL, 1994), supported by component testing, has been used to find suitable geometries for the top plate in compression with the assumed boundary conditions provided
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by the web. The plate thickness, the material grade, the positions of the intermediate and edge stiffeners were finally selected to ensure that the elastic buckling stress of the top plate exceeded the yield stress of the plate, which was assumed to be 350 MPa. A more detailed description of the finite strip analysis and the model used is given elsewhere (Gläsle et al., 2004). The deep longitudinal intermediate stiffeners are rolled into the top plate and a simple lip stiffener is formed along each longitudinal edge to increase the flexural stiffness and compressive capacity of the top plate. The top plate is produced in four finished widths up to nominally 200 mm and varies with the section heights, i.e. TD 90/TD 110, TD 140/TD 160, TD 210 and TD 260. Depending on the panel width, the position of the longitudinal stiffeners changes. The top plate is normally made out of inexpensive uncoated black steel of a lower grade (G250) and is available in different base metal thicknesses, ranging from 1.5 mm to 3.0 mm. This can result in cross-sectional areas equivalent to up to three 16 mm diameter reinforcing bars which is not only utilised at the formwork stage but also in the final composite slab.

2.3.4 Mechanical connectors

To establish the mechanical connection between the three basic components of the cellular section, viz. top plate, webs and base panel, round press-joints (TOX) are used. Press-joining is a joining method without auxiliaries, i.e. it does not need consumables such as bolts or rivets, and connects the material on the basis of local material deformation. The method is referred to as clinching. Since the joining process of TOX joints does not involve cutting of the material, the galvanised coating on the base panel and the web remains undamaged, resulting in a watertight connection. The TOX joints are nominally 8 mm in diameter and form a strong mechanical connection between the components of the panel. They are spread along the panels at regular predetermined centres (up to two uniform spacings are possible within the length of the same panel), and concentrated at the ends of the panels (typically four joints at a spacing of 20 mm) where the support reaction is applied. The protuberance of the TOX joint faces upwards on the base panel, and downwards on the top plate, thus not interfering with the use of the panel. The regular indents formed by the TOX punch on the underside of the base panel are a visually distinctive feature which differentiates the hybrid steel deck from other composite steel decks. A more comprehensive description of the mechanical connectors and their historical background will be given in Chapter 2.4.

2.3.5 End or internal diaphragms

Diaphragms are essential at the ends of the panels where they fall on supports to prevent several failure modes, e.g. local buckling of the webs, particularly in the lower portions where the support reactions are concentrated. The thin-gauge open bracket-shaped diaphragms allow concrete to freely flow into the end regions of the panel when a solid slab is required for these regions, since holes are not normally punched closer than 200 mm clear to the panel ends. They also allow hand access down the panels for installing services such as lighting. Self-piercing rivets (HENROB) are used to connect the diaphragms to the webs and the base panel. In certain applications, e.g. panels with a cantilever, internal diaphragms may be required.
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2.3.6 Infill panel

Infill panels of various shapes can be fitted alternately between the main hybrid steel deck panels, resulting in an improved economy for shorter-spanning slabs by halving the number of main panels. The infill panels can also be trimmed to width so that the decking fits within the boundary of the construction. The current infill panel is very similar in shape to the base panel except that the lap joints have been modified to allow the infill panel to simply “hang” between two main panels and that both overlapping ribs have dimple embossments.

Using this modular approach, a wide variety of forms is possible. Figure 2-6 shows four different arrangements of the hybrid steel deck, with group I and II employing only the main hybrid steel deck panels whereas alternating infill panels are used in group III and IV. For group I and III, the webs of the main panels are holed to create a solid slab. In contrast, the cellular sections form a void for group II and IV by not hoiling the webs and blocking the ends of the main panels (Komselis et al., 2005a).

![Figure 2-6. Different arrangements of the hybrid steel deck system](image)

2.3.7 Examples of commercial applications

To date, the hybrid steel deck system (TRUSSDEK II) has been successfully used in a variety of different commercial projects within Australia. Selected examples of design and construction experience can be found elsewhere (Komselis et al., 2005b). Figure 2-7 to Figure 2-10 give some impressions of the versatile use of the hybrid steel deck system.

![Figure 2-7. Truss-Truss arrangement (left) and Truss-Infill arrangement (right)](image)
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2.4 MECHANICAL CONNECTORS

2.4.1 General

Numerous methods for joining thin-walled elements have been developed (Hahn et al., 2000). Generally, these joining methods can be divided into two categories, viz. joining methods with or without the need of auxiliaries. The methods using auxiliaries differ in the area required for the connection where adhesives are used to create a surface connection, or more conventional consumables (such as bolts or rivets) are used to create a point connection. In recent years, joining methods without the need of auxiliaries became more popular, in particular in the automotive and white goods industries. The method is based on local plastic material deformation and therefore represents an interface between joining technology and the technology of metal forming. This joining
process is also known as “clinching” since the different element layers are locked or clinched together by means of local forming or cutting (Muraski, 1990).

The first patent pertaining to clinching was granted as early as 1897 in Germany (Thies, 1897). In the formation of a connection, the punch (which could be round, polygonal or oval) was pushed into the sheets to be connected causing material to be extruded through to the other side. The extruded material was then compressed, forming a button that had a diameter greater than the punched hole, as can be seen in Figure 2-11.

Figure 2-11. First patent pertaining to clinching (Thies, 1897)

Although the principles of clinching have been available for almost a century (Varis, 1998b), its potential was not fully appreciated on a large industrial scale until the 1980’s when it was re-discovered for the automotive industry. DAIMLER-BENZ in Germany was the first company to introduce clinching to series production of cars in 1980 (Zeitmann, 1996). Since then, the developments have been rapid and considerable research was carried out to investigate this innovative type of connection, resulting in a large variety of different clinching systems (Hahn and Budde, 1990). It then followed in 1985 that the German Standard DIN 8593, Part 5 “Manufacturing Process Joining - Intersperse Joining” started to categorize and define the clinching technology (Liebig and Beyer, 1987). In 1987, the TOX GmbH was founded in Germany and invented an unique single stroke clinch system with a fixed die (known as the TOX system) that received a European patent in 1991 (Rapp, 1991).

In principle, the clinching technology can be divided into two categories (Hahn and Budde, 1990), viz. joints with the need for cutting (shear clinching) and joints without the need for cutting (press clinching). If the joint is obtained by the successive operation of several tools, e.g. one tool for cutting and one tool for forming, the joining process is multi-stage. If the connecting interface is formed during the uninterrupted stroke of a single tool part, the joining process is single-stage. Consequently, all press clinching systems are single-stage processes either with a split die or a fixed die. The idea behind press clinching is to increase the joint strength by reducing the cutting for the benefit of material forming. Clinch connections can come in a variety of different shapes where oblong (S- and H-type shear-insertion join system) or star-shaped (Clinch-system) joints are common for shear clinching systems. By using a round tool, which is typical for press clinching systems (Tog-L-Loc system, Rivet system, TOX system), the strength is nominally equal in all directions in the plane of the sheets. An overview of the various joining methods for thin-walled elements, including selected examples for each joining system, is given in Figure 2-12.
Since the TOX system has been chosen to establish the mechanical connection between the components of the main hybrid steel deck panel, this clinching technology is more closely described in the following.

### 2.4.2 Manufacturing process of TOX joints

In order to obtain a round TOX press-joint, a simple punch presses the materials to be joined into a fixed die cavity. As the force continues to increase, the punch side material is forced to spread outwards within the die side material (TOX Pressotechnik, 2005). Figure 2-13 illustrates various stages of the manufacturing process.

The result is a button-shaped joint that interlocks the two materials to be connected. The characteristic form of the joint is mainly determined by the shape of the die. A standard code system has been developed to describe the shape of the manufacturing tools (Varis, 2000). For the die tool,
the following codes are used: BB, BC and BD, which differ in the depth of the ring groove on the bottom of the die (BB for the deepest, and BD for the shallowest ring groove). The two letters are followed by a four to five digit number where the initial two or three numbers refer to the diameter of the TOX point (in mm/10) and the last two numbers signify the depth of the die mould (in mm/10). For the punch tool, only two standard shapes exist: A (for a conical punch) and AB (for a straight-sided punch). These letters are followed by a four to six digit number where the initial two or three numbers refer to the diameter of the punch neck (in mm/10) and the last two or three numbers denote its corresponding length (in mm/10). Apart from these standard shapes, many more special shapes exist for particular cases.

A typical cross-section with some characteristic designations, as well as a micrograph of a round TOX joint, is given in Figure 2-14. The neck thickness is the thinnest part of the extruded material on the punch side and plays an important role in the shear strength of the joint. The pull-out strength of the joint is mainly influenced by the size of its undercut which is the interlock between the joining parts. The quality of a joint made with a fixed die (such as a TOX joint) can be evaluated in a non-destructive way by measuring the bottom thickness in the middle of the base of the joint, also known as the “X-dimension” (Varis, 2000), and comparing it to the manufacturers’ recommendations. This typically allows a tolerance of 15% (TOX Pressotechnik, 2005). For the specimens tested in this study, a digital measuring caliper (ODITRONIC EDO1025) was used to measure the TOX bottom thickness. Generally, a larger X-dimension results in a higher shear strength whereas a smaller X-dimension results in a higher pull-out strength of the clinched connection (Liebig et al., 1989), and a procedure to find the optimal clinching parameters exists (Varis, 2000; Varis and Lepistö, 2003). In the case of the hybrid steel deck with many different possible material combinations, the various TOX manufacturing tool sets were kept to an acceptable minimum, resulting in sometimes non-optimal connections for the materials given. Following are the manufacturing tools used to produce the specimens tested in this research study: A60100/BB8012 for the TOX joints in the base panel, and A52100/BC8020 or AB52100/BD8014 for the TOX joints in the top plate.

A purpose-designed press has been built to connect the individual components of the main hybrid steel deck panel and form a cellular section (see Figure 2-15). The press station carriage, which contains four press units working simultaneously, moves along the specimen, driven by an electric step motor. Two standard oil cylinders are used for the bottom part and two for the top part, each mounted on a CEB-type press frame. Each cylinder has an adjustable electronic oil pressure switch to
control the return stroke signal after the required pressure is reached. A linear relation between oil pressure and press force exists to form a joint. Results of preliminary tests indicated that the ultimate strength of the TOX connectors may be affected by the applied oil pressure. In particular, thinner materials appeared to be more affected than thicker materials, noting that the measured X-dimension was within the manufacturers’ tolerance. It is therefore important that the applied oil pressure can be adjusted during manufacturing to ensure appropriate press forces and quality joints. However, up to this date, this could not be achieved with the thinnest base panel material, viz. 0.55 mm, which is therefore currently not used in actual applications.

![Figure 2-15. Press station carriage containing the four press units](image)

### 2.4.3 Advantages and disadvantages of TOX joints

Clinching, in particular press clinching, has many advantages compared to other joining methods (e.g. screwing, riveting or spot welding) and is preferred when conventional joining techniques reach their limits technologically and/or economically. The advantages of clinching are well recognized in the automotive and white goods industries, and in recent years also in the building industry, especially for steel-frame houses (Pedreschi and Sinha, 1996). The following list provides a selection of the most significant advantages of the TOX system:

- No need for consumables (e.g. bolts, screws, etc.) and therefore saves costs and weight
- Suitable for dissimilar materials (e.g. different thicknesses and/or grades)
- Suitable for materials that are difficult to weld (e.g. galvanized steel, aluminium, etc.)
- No special preparatory work needed (e.g. pre-drilling, surface cleaning/roughening, etc.)
- No post-processing work needed and can be used immediately
- Manufacturing process has a very good repeatability
- Does not destroy protective coatings (e.g. galvanising, paint, etc.) therefore no corrosion/rust
- Does not leave sharp edges, burrs or metal chips behind
- Uniform planar shear force capacity due to circular shape of joint
- Higher dynamic strength compared to spot welding
- Suitable for air- and watertight joints
- No thermal processing and therefore environmentally friendly with no fumes or flashes
Chapter 2. Development of an ultra long-spanning steel deck

- Fast and efficient technology (multiple connections can be made simultaneously and taking only seconds to form a joint and move to the next position)
- Low-noise manufacturing process
- Low energy and investment expenditure
- Suitable for high automation in factory production
- Minor wear and tear of the tool sets (between 200,000 and 400,000 joining points can be expected during the life of each tool set)

Despite the many advantages that clinching has over other joining methods, there are also some disadvantages that should be noted. The static shear strength of a TOX joint is typically in the range of between 50 to 70 percent compared to that for a similar spot weld and, as for spot welding, the clinching tools need to have access to both sides of the sheets to be joined. Since the equipment required for efficient clinching is rather heavy, the use of clinching on building sites is limited and factory production is favourable. The quality of clinch connections can be very sensitive to the presence of lubricants (Liebig and Mutschler, 1994) which can be critical, in particular for TOX joints with a fixed die, since the volume of a die with a diameter of 8 mm is about 0.1 ml. If this space is partially or fully filled with lubricant, the forming operation can be disrupted (Liebig et al., 1989) and result in a faulty joint. Generally, the quality of clinch connections is not as obvious to determine as for connections using auxiliaries, such as bolts or screws. However, a simple and non-destructive way is to compare the joints’ geometry to the manufacturers’ recommendations. For a more systematic quality assurance, the development of press force during manufacturing of the joint can be automatically monitored and compared to established values (Liebig et al., 1995).

2.4.4 Additional characteristics of TOX joints

During the process of press clinching, the material within the joint undergoes significant local plastic deformations, resulting in strain hardening of the material and an increase in the strength of the joint. This effect has been studied on a TOX joint (Liebig et al., 1989) using sheet metal with an initial hardness of 93/110 HV0.1 (Hardness Vickers).

![Figure 2-16. Hardness distribution of a TOX joint (Liebig et al., 1989)](image-url)
Figure 2-16 shows the hardness distribution along three measurement lines through the joint. The greatest hardness was found on the punch side material in the neck of the joint with values two and a half times higher than the initial hardness. High hardness values were also found around the interface near the centre of the joint, and in the area in line with the interface on the die side material. Methods to convert the measured hardness values into the corresponding yield stress exist (Tekkaya, 2001).

During the clinching process, severe cold-forming takes place in the vicinity of the joint. The localised plastic deformation introduces residual stresses in the surrounding materials which may affect the mechanical behaviour of the joint structure. This might be even more significant for adjacent joints where the stress fields overlap. While high tensile residual stresses have been reported in spot welds, only little knowledge about the residual stress state in clinched connections exists (Gibmeier et al., 2002; Lin Peng et al., 2003). The investigation comprised TOX joints using microalloyed steel with a yield strength of 363 MPa. The results indicated a low level of mean residual stresses, with similar values in both the punch side and die side material. Predominantly compressive residual stresses were measured inside the joint and in the immediate vicinity of the joint (up to 6 mm outside the clinch diameter) which were generally balanced by tensile residual stresses with increasing distance from the joint.

Since the clinching process involves significant plastic deformation of the material in the vicinity of the joint, material with sufficient ductility is needed to avoid fracture of the material during the formation of the joint. Increasing the strength of structural steel typically results in a reduction of its ductility which consequently can reflect on its suitability for clinching. Although considerable research has been undertaken into clinching of mild steel, only little has been published concerning the suitability of clinching for high-strength structural steels (Hahn and Schulte, 1995; Liebig and Bober, 1986; Varis, 1998a; Varis, 2003). As a general rule, materials able to be bent 180° with zero radius can be clinched (Varis, 2003). A more tangible rule established the following limiting values: elongation at failure ($A_{80}$) greater than ten percent and a 0.2 % proof stress less than 550 MPa (Varis, 1998a). However, in an extensive series of single lap shear tests, even high-strength structural steel not fulfilling these limiting values ($A_{80} = 3.6 %; 0.2 %$ proof stress greater than 800 MPa) could successfully joined by clinching including TOX joints (Varis, 2002). The results of this study, and preliminary testing at TOX SYSTEMS Pty. Ltd. in Melbourne Australia on the suitability of TOX press-clinching for high-tensile galvanized G550 sheet steel, gave confidence in the potential use of this material for the base panel of the hybrid steel deck.
CHAPTER 3
BASIC BEHAVIOUR OF TOX JOINTS

3.1 SHEAR STRENGTH OF ROUND TOX PRESS-JOINTS

3.1.1 General

Round TOX press-joints are a key element in the production of the hybrid steel deck since they connect the individual components into a cellular section. The hybrid steel deck is the first known composite deck that utilises clinch connectors as a structural, i.e. load carrying, element and its success depends on the effectiveness of the joints. Although rectangular shear clinches, i.e. joints with the need of cutting, have already been used in the building industry (mainly for light-frame housing), the use of round press-joints, i.e. joints without the need of cutting, was preferred for a composite structural deck where the usage of high-strength steel and leakage of concrete could be a potential problem. Even though the manufacturing process of a clinch connector is relatively fast with about one second per set of joints, the amount of joints should be kept to a minimum to allow higher production rates of the composite deck. Hence, the knowledge of the mechanical connectors’ shear strength plays an important role in assessing the overall performance of the hybrid steel deck.

Rectangular shear clinches

Although a significant amount of research has been undertaken into the behaviour of clinch connectors, the main focus in the early days was on joints used in the automotive and white goods industry, which were pre-dominantly rectangular shear clinches. Many tests were carried out to determine the shear strength of this new connector type, and it was soon found that the cohesion of the joints is more sensitive in regards to base material strength than to its thickness (Liebig et al., 1984). In 1988, an extensive research project with over 600 shear tests was carried out at the University of Karlsruhe in Germany (Baehre and Becker, 1988). The investigation comprised materials ranging from 0.5 to 1.5 mm thickness, as well as different orientations of the joint relative to the applied load (transverse and longitudinal). Based on the test results, a design equation for sheets of equal thickness
was established. However, it was also pointed out that it was only valid for optimized connections and materials used in this investigation:

\[ P = f b t \beta \]  \hspace{1cm} (3-1)

where \( b \) is the width of the joint, \( t \) the sheet thickness, and \( \beta \) the tensile strength of the material. The orientation of the joint is expressed in the factor \( f \), and highly dependent on the type of manufacture the joint. No design equation was found for specimens with different sheet thicknesses, but it was noticed that the shear strength is more dominated by the thickness of the punch side material.

In the early 1990’s, an increase use of cold-formed steel structures as primary structural elements in building construction was observed (Lawson, 1992), but the advantage of using cold-formed structures was often offset by the costs of fabrication which made alternative joining methods more attractive. Up to this time, clinch connectors were predominantly used in the sheet metal work industries and little was published on their behaviour and strength. For this reason, the University of Edinburgh, UK set up a comprehensive research program to examine the behaviour and use of rectangular shear clinches in the more critical area of building structures. In a first test series (Davies et al., 1996), the focus was on different types of steel, a range of material thicknesses, and various connector orientations in regard to the applied load. A total of 39 shear tests were carried out on specimens with sheets of equal thickness, ranging from 1.5 to 2.0 mm. Based on a line of best fit for all test results, the following expression was established:

\[ P = (5.63 - 0.0265 \alpha) (f_u^{0.98} t^{1.45}) \]  \hspace{1cm} (3-2)

where \( \alpha \) is the applied load angle, \( f_u \) the tensile strength of the material and \( t \) the sheet thickness. It should be noted that this equation does not recognize the actual joint geometry and therefore is only applicable for similar shear clinches. In a second series (Davies et al., 1997), the behaviour of groups of rectangular shear clinches subjected to pure bending moment were studied more closely. It was found that the precise orientation of the connectors around the centroid had a marked effect on the moment capacity and that the group of connectors exhibit varying degrees of plasticity. The structural behaviour of full-scale structures was part of a third series (Pedreschi et al., 1997). Single span beam tests indicated that substantial end moments can be developed, whereas full-scale roof truss tests proved that joints of sufficient strength can be produced so that failure occurred in the section rather than in the connection.

In the year 2000, an innovative approach to describe the shear strength of rectangular shear clinches was made at the Technical Research Centre of Finland (Helenius, 2000). Rather than carrying out new shear tests, the focus was on developing analytical design equations and comparing them to existing test results (Lu et al., 1998; Varis, 1998a). The approach recognizes the fact that different failure characteristics exist for different orientations of the joint relative to the applied load, resulting in design equations for loading in longitudinal and transverse direction:
Chapter 3. Basic behaviour of TOX joints

Longitudinal: \[ P = t_{\text{hor}} b f_u \]
Transverse: \[ P = 2 t b r_u = 2 t b (f_u / \sqrt{3}) \] (3-3)

where \( t_{\text{hor}} \) is the horizontal projection of the thickness of the upper sheet at the bottom, and \( r_u \) the ultimate shear strength which can be obtained by dividing the ultimate tensile strength \( f_u \) by \( \sqrt{3} \) according to the von Mises’ failure condition. Although the models show good agreement with the test results, it was admitted that the formula for loading in the longitudinal direction cannot be used in practice since it requires knowledge of the internal geometry of the clinch.

Just recently, the University of Edinburgh, UK published an alternative method to predict the shear strength of rectangular shear clinches (Pedreschi and Sinha, 2006) which also takes account of the different failure modes in the longitudinal and transverse direction. For a joint orientated in the longitudinal direction, the shear load is predicted by considering the mechanism to cause the initial distortion of the punch side part of the clinch by forming plastic hinges. For a joint orientated in the transverse direction, the shear load is the product of the nominal cross-sectional area resisting shear, the yield strength of the material used and an experimentally obtained factor \( \phi \) which accounts for changes in both thickness and steel strength in the vicinity of the clinch:

Longitudinal: \[ P = 1.74 t f_y [2 + 1.15 t / (2 + 0.207 t)] \]
Transverse: \[ P = \phi f_y 2t w, \quad \text{with} \quad \phi = 0.52 + 0.3 t \] (3-4)

where \( f_y \) is the yield strength of the steel, \( t \) the thickness of the material on the punch side and \( w \) the width of the punch of the joining tool.

Round press clinches

While over the last two decades most effort went into researching rectangular shear clinches, it appears that round press clinches were largely neglected. In 1989, the first report on the shear behaviour of round TOX press-joints was published (Liebig et al., 1989) which revealed an increase of shear strength with the use of thicker material, noting that only specimens with the same material thickness on punch and die side were tested. A comparison between different sized press-joints showed that joints of 8 mm diameter could achieve higher shear forces than 6 mm joints. The report also included tests on different types of material, resulting in approximately 25% higher shear forces for electronically galvanized and hop-dipped aluminized steel sheets compared to uncoated steel sheets. This effect was explained by the fact that these type of coatings tend to cause local cold working during manufacture or when undergoing shear loading. Since then, many shear tests have been performed at TOX PRESSOTECHNIK to provide guidance for the selection of proper tool sets (TOX Pressotechnik, 2002). The tests cover a wide range of different material types and thicknesses, but were carried out individually and show no particular systematic pattern. However, evaluating the results indicate higher shear loads for specimens with larger joint diameters, as well as for specimens with thicker punch side material. Reduced shear loads were noticed if the material on the die side was thicker than on the punch side.

In 1998, a study on 4.5 mm round press-joints and rectangular shear clinches was carried out at the Helsinki University of Technology (Lu et al., 1998). Based on the test results, design values for the ultimate shear strength were determined for each material combination tested and type of joining
method, but no design equation was established. However, the study included an interesting series of shear tests with corroded specimens. The specimens were exposed to a cyclic corrosion condition (wet/dry/humidity) for 840 hours and the results showed no change in shape of the load-displacement curve compared to the shape of their untreated counterpart. In fact, the maximum shear load for zinc-coated material was about 9% higher after corrosion than before and it was assumed that corrosion made the specimen connect much more tightly and therefore the effect of friction was enhanced. In 1999, another interesting study was presented at the 4th International Conference on Steel and Aluminium Structures in Helsinki, Finland (Kolari, 1999) which investigated the load-sharing relationship between fasteners in multiple fastener connections, including circular press-joints. In general, the capacity of multiple fasteners is less than the sum of a single fastener connection capacity and the capacity depends on how efficiently the fasteners can redistribute the applied force (Macindoe and Hanks, 1994). It is known that load sharing mainly depends on the fastener’s ability to deform adequately, but deformation capacities of 0.4 mm to 3 mm for the tested press-joints were considered to be insufficient. For this reason, load sharing tests were necessary to ensure the applicability of press-joints in building industry products. Connections with three to five fasteners in a row were tested and results showed that the capacity of connections with multiple press-joints was close to the sum of the individual connector capacity, indicating that press-joints do share load efficiently despite their low deformation capacity.

As part of a research project to develop analytical design equations for clinch connectors, round press clinches were also under investigation at the Technical Research Centre of Finland (Helenius, 2000). To the author’s knowledge, this is the first attempt to establish a design model for the shear strength of round press clinches. The model assumes that the narrow neck of the material on the punch side will be plastified first due to localized pressure on a limited contact area. The concept of contact areas to transmit forces through an interface was already earlier developed to describe the failure mechanism of clinch connectors (Gao and Budde, 1994). Provided the joint is ductile enough, the entire neck will reach a plastic state by increasing the pressure and the shear strength can be calculated by multiplying the cross-sectional area of the neck by the ultimate shear strength $\tau_u$ of the material. The ultimate shear strength may be obtained by dividing the ultimate tensile strength $f_u$ by $\sqrt{3}$.

$$P = \pi (d+t_{\text{min}}) t_{\text{min}} \tau_u = \pi (d+t_{\text{min}}) t_{\text{min}} (f_u/\sqrt{3}) \quad (3-5)$$

where $d$ is the diameter of the punch and $t_{\text{min}}$ the smallest thickness of the neck on the punch side. The equation was then used to predict the shear strength of tests carried out at the Helsinki University of Technology (Lu et al., 1998) and the Technical University of Lappeenranta (Varis, 1998a), but showed a rather poor agreement between the experimental and predicted values. For most of the tests, the ratio was approximately two, which was explained by the effect of high strain hardening of the material during the formation of the clinch but could not be accounted for in the design equation. It was also pointed out that the model cannot be used in practice since knowledge of the internal geometry of the clinch is required.
In a more recent study (Di Lorenzo and Landolfo, 2004), a design function is proposed in order to assess the shear strength of circular press-joints. In this model, the circular press-joint is treated similar to a tubular rivet whose collapse occurs by crushing of the fastener against the sheet.

\[ F_{rd} = \alpha_c f_u d_p t_p / \gamma_{M2} \]  

(3-6)

where \( \alpha_c \) is the clinching bearing coefficient, \( f_u \) the ultimate strength of the material, \( d_p \) the internal diameter of the clinch measured on the punch side, \( t_p \) the sheet thickness on the punch side and \( \gamma_{M2} \) the partial factor for calculating the design resistance of the mechanical fasteners. The clinching bearing coefficient was obtained experimentally and represents solely the press-joint under investigation, noting that tests were carried out with one material thickness only and results are not applicable for other combinations.

**Testing**

Because of the difficulty in applying previous research models to practice, particularly for the range of thicknesses and steel grades planned to be used in the hybrid steel deck, it was considered that testing had to be carried out. The tests were intended to give a better understanding of the shear behaviour of the round TOX press-joints when dissimilar material combinations in regards to thickness and steel grades are used. In the following, the results of 42 shear tests carried out in two series at the University of Western Sydney are presented.

### 3.1.2 Test setup

In 1996, new standard tests for single-point fasteners were introduced into the Australian Cold-Formed Steel Standard (AS/NZS 4600, 2005) which enables the performance of different types of single-point fasteners to be compared. The standard tests followed recommendations (Macindoe and Hanks, 1994) which proposed the single lap shear test and cross-tension test after reviewing overseas standards and recommendations. The single lap shear test was chosen because it was the most popular test for determining the shear characteristics of the connection since the test specimen is easy to construct and test and often reflects the actual application of the fastener well. Even though the Australian Standard recommends these tests also for clinch connectors, it was believed that a modified test set-up would better simulate the behaviour of the press-joints in the hybrid steel deck. A single lap shear test allows tilting of the fastener (Stark and Toma, 1982) which can result in considerable distortion of the thin sheet material in the case of clinch connectors, as illustrated in Figure 3-1.

![Figure 3-1. Tilting of single point connectors](image)
The rotation introduces tension forces on the connector which potentially reduces the ultimate load in pure shear. For the hybrid steel deck, such a local rotation is not possible, in particular when the base panel is in tension and the web maintains the base panel in a flat condition.

To overcome this problem, a back-to-back test set-up was chosen to perform the first series of tests. The test set-up is similar to the single lap shear test, but uses two identical test specimens with the sheets of the punch side back-to-back to each other (Figure 3-2). The idea behind using two specimens was to eliminate the rotation when the sheets press against each other. Tests were carried out under displacement control with a constant cross-head speed of 0.2 mm/min (except for the 1.6 mm HA70T material, which was tested at double speed) in an INSTRON universal testing machine (Model No. 6027), equipped with the standard 200 kN load cell and calibrated according to the NATA (National Association of Testing Authorities, Australia) specifications. The punch-side of the specimen was held with the standard grips of the testing machine whereas the jaw on the die side of the specimen was replaced with a metal clevis to avoid undesirable bending of the sheets when tightening the jaw. The sheets on the die side were therefore holed and clamped between two 8 mm thick metal plates using a M10 bolt. Since a back-to-back arrangement was used, a metal packer of equal thickness to the two sheets on the punch side was placed between the two sheets to fill the space between them. The plates were then attached to the testing machine via a 24 mm bolt, forming a pin-ended connection. Two standard 50 mm gauge-length extensometers (INSTRON 2630–036 and –038) were attached directly to the specimen at the overlapping region to measure the deformation of the TOX connector. The deformations, together with load and crosshead movement, were recorded using the software package MERLIN installed on a personal computer. The back-to-back test set-up, as shown in Figure 3-2, was used to investigate 6 mm and 8 mm round joints as well as specimens with three connections in a row.

Although the back-to-back test set-up considerably reduced rotation of the sheets, twice as many test specimens had to be made since two specimens were tested simultaneously and only the total load
acting on both specimens could be measured. Therefore, a modified test set-up with a single lap specimen was used for the second series of tests (Figure 3-3) where the influence of the thickness of the punch side material was more closely investigated. To prevent rotation of the sheet, an 8 mm thick plate was located on the punch side, and a 50x50x4 mm plate on the opposite side loosely clamped to it. The punch-side of the specimen together with the 8 mm plate was held with the standard grips of the testing machine whereas the sheet on the die side was clamped in the same way as in the back-to-back test set-up. Since only a single specimen was used, a packer was not needed.

3.1.3 Specimen details

The dimension of the test specimen followed the suggestions made in (AS/NZS 4600, 2005), which specifies 50 mm lap length and a minimum of 150 mm unclamped length of the specimen with a width of 50 mm. The specimens in the first test series therefore had a width of 50 mm and an overall length of 450 mm and 400 mm for the specimens with three connections in a row and single connector, respectively (Figure 3-4). About 90 mm at each end of the specimen was dedicated for clamping to the testing machine. With a connector spacing of 25 mm, the length of the overlapping region of the two sheets was 50 mm for a single connection and 100 mm for three connectors in a row.

For the second test series, the specimen dimensions were slightly modified to overcome difficulties in measuring the deformation of the TOX connector with the 50 mm extensometers. Research has demonstrated that the dimensions of the specimens used in lap shear tests have no significant influence on the shear strength of the clinch (Di Lorenzo and Landolfo, 2004). The overlapping ends of the specimen were tapered and the width increased to 65 mm (Figure 3-5). With this geometry it was possible to position one extensometer edge exactly on the centre line of the connector, thus eliminating the effect of material elongation on the punch side of the specimen. It also reduced the possibility of one extensometer falling off. The width of the sheet on the punch side had to be reduced to centrally place the specimen into the 50 mm wide INSTRON standard grips.
At the time the tests were conducted, the hybrid steel deck was still in the planning stage and the TOX machine, as described in Chapter 2.4.2, was not available. Prefabricated blanks of sheet metal were therefore shipped to TOX SYSTEMS Pty. Ltd. in Melbourne Australia to connect various material combinations with different types of press-joints. In the first test series, 26 tests were carried out to investigate the difference between 6 mm and 8 mm round joints as well as the influence of having three connections in a row. Another 15 specimens were tested in the second series where the influence of the thickness of the punch side material was more closely investigated. All press-joints in this investigation were made with a standard TOX ‘Powercylinder’ press using the tool sets listed in Table 3-1.
Table 3-1. TOX manufacturing tool sets

<table>
<thead>
<tr>
<th>First test series</th>
<th>Material punch side</th>
<th>Material die side</th>
<th>Diameter</th>
<th>Punch No.</th>
<th>Die No.</th>
<th>Press force</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.55 mm (ZHT G550)</td>
<td>0.70 / 0.90 mm (G300)</td>
<td>6 mm</td>
<td>A42100</td>
<td>BB6010</td>
<td>41 kN</td>
</tr>
<tr>
<td></td>
<td>0.55 mm (ZHT G550)</td>
<td>0.70 / 0.90 mm (G300)</td>
<td>8 mm</td>
<td>A62100</td>
<td>BB8010</td>
<td>52 kN</td>
</tr>
<tr>
<td></td>
<td>1.60 mm (HA70T)</td>
<td>0.70 / 0.90 mm (G300)</td>
<td>8 mm</td>
<td>ABS2100</td>
<td>BD8014</td>
<td>52 kN</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Second test series</th>
<th>Material punch side</th>
<th>Material die side</th>
<th>Diameter</th>
<th>Punch No.</th>
<th>Die No.</th>
<th>Press force</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.55 mm (ZHT G550)</td>
<td>0.70 / 0.90 mm (G300)</td>
<td>8 mm</td>
<td>A60100</td>
<td>BD8012</td>
<td>54 kN</td>
</tr>
<tr>
<td></td>
<td>0.75 mm (G550)</td>
<td>0.70 / 0.90 mm (G300)</td>
<td>8 mm</td>
<td>A60100</td>
<td>BB8012</td>
<td>54 kN</td>
</tr>
<tr>
<td></td>
<td>1.00 mm (G550)</td>
<td>0.70 / 0.90 mm (G300)</td>
<td>8 mm</td>
<td>A60100</td>
<td>BB8012</td>
<td>54 kN</td>
</tr>
</tbody>
</table>

Various tool set combinations were previously investigated at TOX SYSTEMS Pty. Ltd., to ensure satisfactory strength capacities of the joints for all material combinations by using a minimum amount of tool set combinations. For instance, the same tool set is used for all material combinations in the second test series with the exception of the die tool for the 0.55 mm punch side material. Characteristic dimensions of the TOX joint, such as the remaining bottom thickness \( x \) of the two connected sheets, the inside diameter \( i \) on the punch side and the outside diameter \( o \) on the die side, are illustrated in Figure 3-4 and Figure 3-5 and measurements are given in Table 3-2.

Each specimen was given a distinct alpha-numeric code, determined from basic information of the specimen. The initial letter represents the test set-up (A for the back-to-back, and B for the modified test set-up), followed by two numbers which state the amount of connectors in a row and their nominal outside diameter in millimetres. The following three numbers represent the nominal thickness of the punch side material, followed by two numbers which represent the nominal thickness of the die side material. Both thicknesses are given in millimetres times 100. The final number, separated by a dash, states the specimen number. Examples of the code are given below.

Table 3-2. Average TOX dimension

<table>
<thead>
<tr>
<th>First test series</th>
<th>Code</th>
<th>( x ) (mm)</th>
<th>( i ) (mm)</th>
<th>( o ) (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A16-05570-1,2,3</td>
<td>0.35</td>
<td>4.2</td>
<td>6.2</td>
</tr>
<tr>
<td></td>
<td>A36-05570-1,2,3</td>
<td>0.45</td>
<td>6.2</td>
<td>8.1</td>
</tr>
<tr>
<td></td>
<td>A18-05570-1,2,3</td>
<td>0.90</td>
<td>5.1</td>
<td>8.1</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Second test series</th>
<th>Code</th>
<th>( x ) (mm)</th>
<th>( i ) (mm)</th>
<th>( o ) (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A18-05570-1</td>
<td>0.45</td>
<td>6.3</td>
<td>8.2</td>
</tr>
<tr>
<td></td>
<td>A18-07570-1</td>
<td>0.40</td>
<td>6.3</td>
<td>8.2</td>
</tr>
<tr>
<td></td>
<td>A18-10070-1</td>
<td>0.60</td>
<td>6.3</td>
<td>8.2</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Code</th>
<th>( x ) (mm)</th>
<th>( i ) (mm)</th>
<th>( o ) (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A16-05590-1,2,3</td>
<td>0.45</td>
<td>4.3</td>
<td>6.2</td>
</tr>
<tr>
<td>A36-05590-1,2,3</td>
<td>0.55</td>
<td>6.3</td>
<td>8.1</td>
</tr>
<tr>
<td>A18-16090-1,2,3</td>
<td>1.00</td>
<td>5.1</td>
<td>8.1</td>
</tr>
<tr>
<td>B18-05590-1,2</td>
<td>0.55</td>
<td>6.4</td>
<td>8.2</td>
</tr>
<tr>
<td>B18-07590-1</td>
<td>0.50</td>
<td>6.4</td>
<td>8.2</td>
</tr>
<tr>
<td>B18-10090-1,2,3</td>
<td>0.70</td>
<td>6.4</td>
<td>8.2</td>
</tr>
</tbody>
</table>
3.1.4 Material properties

Tensile testing of coupons was carried out to determine the mechanical properties of the material used in the test series. All materials were tested in the longitudinal, i.e. rolling, direction although the testing direction is of minor importance since a round connector was under investigation. Properties of the material were based on the base metal thickness (b.m.t.) although the coupons were tested in their original coated state. A description of the test method and all test results can be found in Appendix A. Average results for the different materials used in this test series are listed in Table 3-3.

Table 3-3. Average tensile coupon test results

<table>
<thead>
<tr>
<th>Punch Side material (G550/HA70T)</th>
<th>t_c (mm)</th>
<th>t_b (mm)</th>
<th>f_y* (MPa)</th>
<th>(\varepsilon_{0.2}^*) (%)</th>
<th>f_u* (MPa)</th>
<th>E* (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>G550/1.00 coated</td>
<td>1.07</td>
<td>0.97</td>
<td>719</td>
<td>0.53</td>
<td>721</td>
<td>229.1</td>
</tr>
<tr>
<td>G550/0.75 coated</td>
<td>0.82</td>
<td>0.74</td>
<td>672</td>
<td>0.54</td>
<td>678</td>
<td>229.2</td>
</tr>
<tr>
<td>ZHT G550/0.55 coated</td>
<td>0.63</td>
<td>0.53</td>
<td>n/a</td>
<td>n/a</td>
<td>771</td>
<td>237.2</td>
</tr>
<tr>
<td>HA70T/1.80 uncoated</td>
<td>n/a</td>
<td>1.62</td>
<td>405</td>
<td>0.46</td>
<td>499</td>
<td>209.5</td>
</tr>
</tbody>
</table>

Values based on b.m.t.

<table>
<thead>
<tr>
<th>Die Side material (G300)</th>
<th>t_c (mm)</th>
<th>t_b (mm)</th>
<th>f_y* (MPa)</th>
<th>(\varepsilon_{0.2}^*) (%)</th>
<th>f_u* (MPa)</th>
<th>E* (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>G300/0.9 coated</td>
<td>0.94</td>
<td>0.88</td>
<td>327</td>
<td>0.39</td>
<td>419</td>
<td>235.7</td>
</tr>
<tr>
<td>G300/0.7 coated</td>
<td>0.82</td>
<td>0.70</td>
<td>343</td>
<td>0.42</td>
<td>418</td>
<td>236.4</td>
</tr>
</tbody>
</table>

Values based on b.m.t.

3.1.5 Test results

During the first test series, various failure modes were encountered depending on the material combination and manufacturing tool set used to form the press-joint. An overview of the principal failure modes, including the failure mechanism, is given in Figure 3-6.

The first mode is a shear failure (mode I) and characterized by a fracture of the interface between the sheets, which is followed by a complete separation of the two sheets. This failure mode typically occurs if the material in the neck on the punch side is insufficient compared to the material on the die side. The second failure mode is a pull-over failure (mode II) where the two sheets separate without fracturing the interface between the sheets due to a loss of the interlock. The neck on the punch side is generally strong enough to carry the applied shear load whereas the sheet on the die side of the joint starts to deform and the joint subsequently opens up. This failure mode can be facilitated by a slight inwards rotation of the sheets on the punch side and could not be observed with the modified second test set-up. Only on a few occasions was a combined shear and pull-over failure (mode III) observed instead of mode I or mode II. In this combined mode, half of the neck on the punch side fractures, while the joint on the die side opens and the remaining parts pull over. Failure mode IV is characterized by a non-separation of the two connected sheets due to a bearing failure of the material on the die side. In this mode, the neck on the punch side can resist the applied shear load without fracturing, in contrast to the material on the die side where the sheet fractures across the joint.
Chapter 3. Basic behaviour of TOX joints

Figure 3-6. Principal failure modes

Results of the first test series, including individual failure modes (separately for specimen A and B in the back-to-back test set-up), are summarized in Table 3-4 for specimens with one connector and in Table 3-5 for specimens with three connectors in a row. For comparison purposes, the ultimate load $P_u$ was also divided by the total number of connectors $n$ resulting in the ultimate load per connector provided that the applied load is distributed equally.
Table 3-4. Test results of first series with one connector in a row

<table>
<thead>
<tr>
<th>Code</th>
<th>n</th>
<th>( P_u ) (kN)</th>
<th>( P_u / n ) (kN)</th>
<th>Mode*</th>
<th>Code</th>
<th>n</th>
<th>( P_u ) (kN)</th>
<th>( P_u / n ) (kN)</th>
<th>Mode*</th>
</tr>
</thead>
<tbody>
<tr>
<td>A16-05570-1</td>
<td>2</td>
<td>4.289</td>
<td>2.145</td>
<td>II / I</td>
<td>A16-05590-1</td>
<td>2</td>
<td>3.833</td>
<td>1.917</td>
<td>I / I</td>
</tr>
<tr>
<td>A16-05570-2</td>
<td>2</td>
<td>4.289</td>
<td>2.145</td>
<td>I / I</td>
<td>A16-05590-2</td>
<td>2</td>
<td>3.945</td>
<td>1.973</td>
<td>I / I</td>
</tr>
<tr>
<td>A16-05570-3</td>
<td>2</td>
<td>3.929</td>
<td>1.965</td>
<td>I / I</td>
<td>A16-05590-3</td>
<td>2</td>
<td>3.655</td>
<td>1.828</td>
<td>I / I</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td></td>
<td><strong>4.169</strong></td>
<td><strong>2.085</strong></td>
<td></td>
<td><strong>Average</strong></td>
<td></td>
<td><strong>3.811</strong></td>
<td><strong>1.906</strong></td>
<td></td>
</tr>
<tr>
<td>A18-05570-1</td>
<td>2</td>
<td>5.845</td>
<td>2.923</td>
<td>II / II</td>
<td>A18-05590-1</td>
<td>2</td>
<td>4.936</td>
<td>2.468</td>
<td>II / III</td>
</tr>
<tr>
<td>A18-05570-3</td>
<td>2</td>
<td>5.762</td>
<td>2.881</td>
<td>II / II</td>
<td>A18-05590-3</td>
<td>2</td>
<td>4.732</td>
<td>2.366</td>
<td>I / III</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td></td>
<td><strong>5.911</strong></td>
<td><strong>2.956</strong></td>
<td></td>
<td><strong>Average</strong></td>
<td></td>
<td><strong>4.834</strong></td>
<td><strong>2.417</strong></td>
<td></td>
</tr>
<tr>
<td>A18-16070-2</td>
<td>2</td>
<td>10.358</td>
<td>5.179</td>
<td>IV / IV</td>
<td>A18-16090-2</td>
<td>2</td>
<td>12.820</td>
<td>6.410</td>
<td>I / I</td>
</tr>
<tr>
<td>A18-16070-3</td>
<td>2</td>
<td>10.089</td>
<td>5.045</td>
<td>IV / IV</td>
<td>A18-16090-3</td>
<td>2</td>
<td>13.098</td>
<td>6.549</td>
<td>I / I</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td></td>
<td><strong>10.127</strong></td>
<td><strong>5.063</strong></td>
<td></td>
<td><strong>Average</strong></td>
<td></td>
<td><strong>12.965</strong></td>
<td><strong>6.482</strong></td>
<td></td>
</tr>
</tbody>
</table>

* Part A / B

Analysing the test results reveals that an 8 mm TOX diameter generally achieves a higher shear load capacity compared to a 6 mm TOX diameter, namely 29 % for specimens with the 0.7 mm die side material and 21 % for the 0.9 mm material. The increase in load capacity is accompanied with a change of failure modes. Specimens with a 6 mm connector mainly experienced a fracture of the neck on the punch side (mode I) whereas a pull-over failure (mode II) was mostly observed for specimens with an 8 mm connector. It appears that the larger volume in the neck of the 8 mm connector is able to resist higher shear loads, thus causing the joint on the die side to open up. By losing the interlock between the sheets, a pull-over failure is inevitable, in particular since the back-to-back test set-up could not fully prevent rotation of the sheets.

In the case of thinner material on the punch side, a reduction of the ultimate shear load by increasing the thickness of the die side material is noticeable, despite keeping the same failure mode. The reduction is higher (18 %) for 8 mm joints compared to 6 mm joints (9 %). Keeping in mind the same TOX tool set was used to produce specimens with 0.7 mm and 0.9 mm die side material, the same inside and outside diameters of the joint were measured. It seems that using a thicker die side material has less influence on the shear failure mode experienced with a 6 mm joint than on the pull-over failure mode observed with an 8 mm joint, suggesting that the size of the undercut of the joint is more affected than the thickness of the neck. This relation cannot be observed in the case of much thicker material on the punch side where a 22 % increase for the 0.9 mm material on the die side was recorded, accompanied with a change of failure modes. Specimens with 0.7 mm die side material experienced a bearing failure of the material on the die side (mode IV) without fracturing the neck on
the punch side. Increasing the thickness of the die side material resulted then in a fracture of the neck on the punch side (mode I) with a complete separation of the sheets. The change of failure mode implies that the joint on the die side is now strong enough to resist even higher shear loads without deforming and opening up, causing the neck on the punch side to fracture. The influence of the die side material thickness $t_d$ on the ultimate load per connector is illustrated in Figure 3-7 for specimens of the first series with one connector in a row.

![Figure 3-7. Influence of die side material thickness on ultimate load (first series)](image)

Typical load-slip curves for the TOX press-joints in the first test series are presented in Figure 3-8 and Figure 3-9, noting that the ultimate load is displayed as load per connector for later comparison reasons. The slip was measured using extensometers over a 50 mm gauge length. Therefore the measurements include not only slip but the extension of the sheets in tension over the length. This extension has been calculated to be small and not significant and hence the extensometer measurements have been taken as the slip of the connector. Similar load-deformation characteristics were observed for tests with the same failure mode and are described below. Results for all tests carried out in this series can be found in Appendix B.

Independent of the failure mode, all specimens show a relative high initial loading stiffness (in the range from 20,000 N/mm to 35,000 N/mm for the 0.55 mm thick punch side material, and from 40,000 N/mm up to 80,000 N/mm for the 1.6 mm thick punch side material) which is a direct result of the press-joint’s mode of performance. Generally, the internal geometry of a press-joint is designed in such a way that it provides a force and form-fit joint between the sheets to be connected. In a perfectly designed press-joint, the force is transmitted without a relative motion within the structure. Depending on the characteristics of the material used, relative motion becomes possible as load is applied due to local overloading of some parts, and consequently reduces the loading stiffness. For example, such a motion is not possible in fused connections, i.e. spot-welding, where the material is melted together...
and the loading stiffness is close to the stiffness of the virgin material (Hahn and Boldt, 1992). On the contrary, the absence of form-fit in a self-tapping screw allows rotation of the screw as load is applied, resulting in a relative low loading stiffness (Lennon et al., 1999).

Figure 3-8. Typical load-slip curve for 6 mm and 8 mm TOX ($t_p=0.55$, $t_d=0.7,0.9$)

Figure 3-9. Typical load-slip curve for 8 mm TOX ($t_p=1.6$, $t_d=0.7,0.9$)

The load deformation response of the tested TOX joints is approximately linear up to about two thirds of the ultimate load (see Figure 3-8 and Figure 3-9) when it starts to curl over and reaches a quasi-plastic state. Larger plastic deformations were observed for 8 mm joints independent of the material thickness on the die side. Slip values at ultimate load were between 0.3 and 0.6 mm for the
Chapter 3. Basic behaviour of TOX joints

0.55 mm punch side material, and around 1 mm for the 1.6 mm material. Soon after reaching the ultimate load, the load drops rapidly in the case of a shear failure of the neck on the punch side whereas for the pull-over failure, the load decreases more slowly and the deformation continues until the joint fully opens and the two sheets separate. The combined shear and pull-over failure shows both characteristics as seen in the load-slip curve of specimen A18-05590-2.

A completely different load-slip response can be observed for the bearing failure mode. After reaching two thirds of the ultimate load, large deformations occur when approaching ultimate load (see Figure 3-9). Once the sheet on the die side starts to fracture across the joint, the load drops slowly until it flattens again due to tearing of the sheet. Usually the test was terminated at this stage.

Tests carried out on specimens with multiple fasteners, in this case three connectors in a row, experienced the same failure modes as specimens with single connectors as seen in Table 3-5. Despite the low deformation capacity of the press-joints, the ultimate load was close to the sum of the individual connector capacity, in fact even slightly higher. For the 6 mm joints, the ultimate load increased by 4.5 % for the 0.7 mm die side material and 0.5 % for the 0.9 mm material. For the 8 mm joints, an increase of 1.3 % was recorded for the 0.7 mm material. Less rotation of the sheets might have been the reason for the higher values.

Results of the second test series, including slip at ultimate load $s_u$ and individual failure modes, are summarized in Table 3-6 for tests undertaken with the back-to-back test set-up and in Table 3-7 for tests undertaken with the modified test set-up. Six different material combinations with an 8 mm TOX diameter were tested. All specimens in the second test series were manufactured with a different tool set to optimise the shear capacity. The punch tool had a slightly smaller diameter and the depth of the die mould was increased. The observed failure modes for each material combination were identical for both test set-ups. However, slightly lower values for ultimate load were recorded for the back-to-back test set-up, suggesting that one connector was loaded more than the other and therefore failed prematurely.

All specimens experienced a shear failure (mode I) except for specimens with the thin 0.55 mm punch side material which mainly failed in a combination of shear and pull-over (mode III). Examining the test results reveals a direct relationship between the thickness of the punch-side material and ultimate shear strength (Table 3-7, Figure 3-10). For combinations with the 0.7 mm die side material, the load increases relative to the 0.55 mm punch side material by 35 % and 75 % for the 0.75 mm and 1.00 mm material respectively. Similar slightly lower gains were recorded for the 0.9 mm die side material, namely 23% and 67 %. It appears that increasing the thickness of the punch-side material strengthens the neck of the joint and therefore results in higher ultimate loads which is usually accompanied with higher amounts of slip.
Table 3-6. Test results of second test series with back-to-back test set-up

<table>
<thead>
<tr>
<th>Code</th>
<th>( n )</th>
<th>( P_u ) (kN)</th>
<th>( P_u / n ) (kN)</th>
<th>( s_u ) (mm)</th>
<th>Mode*</th>
</tr>
</thead>
<tbody>
<tr>
<td>A18-05570</td>
<td>2</td>
<td>5.204</td>
<td>2.602</td>
<td>0.29</td>
<td>III / III</td>
</tr>
<tr>
<td>A18-07570</td>
<td>2</td>
<td>5.930</td>
<td>2.965</td>
<td>0.17</td>
<td>I / I</td>
</tr>
<tr>
<td>A18-10070</td>
<td>2</td>
<td>8.228</td>
<td>4.114</td>
<td>0.46</td>
<td>I / I</td>
</tr>
</tbody>
</table>

* Part A / B

Table 3-7. Test results of second test series with modified test set-up

<table>
<thead>
<tr>
<th>Code</th>
<th>( P_u ) (kN)</th>
<th>( s_u ) (mm)</th>
<th>Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>B18-05570-1</td>
<td>2.725</td>
<td>0.31</td>
<td>III</td>
</tr>
<tr>
<td>B18-05570-2</td>
<td>2.594</td>
<td>0.50</td>
<td>III</td>
</tr>
<tr>
<td>Average</td>
<td>2.660</td>
<td>0.41</td>
<td></td>
</tr>
<tr>
<td>B18-07570-1</td>
<td>3.652</td>
<td>0.62</td>
<td>I</td>
</tr>
<tr>
<td>B18-07570-2</td>
<td>3.535</td>
<td>0.44</td>
<td>I</td>
</tr>
<tr>
<td>Average</td>
<td>3.594</td>
<td>0.53</td>
<td></td>
</tr>
<tr>
<td>B18-10070-1</td>
<td>4.538</td>
<td>0.73</td>
<td>I</td>
</tr>
<tr>
<td>B18-10070-2</td>
<td>4.891</td>
<td>0.68</td>
<td>I</td>
</tr>
<tr>
<td>B18-10070-3</td>
<td>4.527</td>
<td>1.10</td>
<td>I</td>
</tr>
<tr>
<td>Average</td>
<td>4.652</td>
<td>0.84</td>
<td></td>
</tr>
</tbody>
</table>

According to the European Recommendations for Steel Construction (ECCS No.21, 1983), a deformation smaller than 0.5 mm at failure of the fastening is considered to be insufficient to ensure redistribution and equalisation of forces in the connection (brittle failure) and a deformation larger than 3 mm is preferred for load bearing connections. This is clearly not possible for clinches where load is transferred through force and form-fit and the failure is more of a brittle nature. The redistribution and equalisation of forces in press-joints is examined in Chapter 6. Figure 3-10 shows typical load-slip curves for various punch side material thicknesses and a 0.7 mm die side material. The initial loading stiffnesses observed in the tested material combinations are again relatively high and in the range from 20,000 N/mm to 40,000 N/mm.

![Figure 3-10. Typical load-slip curves for 8 mm TOX (\( t_p=0.55,0.75,1.00, t_d=0.7 \))](image-url)
As already observed in the first test series, thicker die side material reduces the ultimate shear load provided that the same TOX tool set was used to produce specimens with 0.7 mm and 0.9 mm die side material. This suggests that the neck of the joint is thinned. The reduction for the 0.9 mm die side material is 2 %, 10 % and 6 % for the 0.55 mm, 0.75 mm and 1.00 mm punch side material respectively. Figure 3-11 illustrates this relation.

![Figure 3-11. Influence of die side material thickness on ultimate load (second series)](image)

### 3.1.6 Conclusions

As previous research on the behaviour of circular press clinches has shown, the shear strength of the joint is highly influenced by its forming process and therefore a direct result of the used tool set for a particular combination of joint diameter, punch/die side material thicknesses and their material properties. This makes a generalization very difficult and conclusions are restricted to specimens with similar characteristics. The tool sets used in this study were optimized to suit the particular needs of the hybrid steel deck and may not be the best solution for other applications. In the first series, four different failure modes were encountered and categorized as shear, pull-over, combined shear and pull-over, and bearing failure. Higher shear load capacities could be achieved with larger joint diameters as well as by increasing the punch side material thickness. Increasing the thickness of the die side material generally resulted in a reduction of ultimate shear load, noting that the same tool set combination was used. Only for a very thick punch side material was the ultimate shear load higher for thicker die side material and can be explained by the change of failure mode. Tests on multiple fasteners showed that the shear load capacity was close to the sum of the individual connector capacity despite the limited deformation capacity of clinch connectors. The load-deformation curve of individual joints is approximately linear up to about two thirds of the ultimate load and shows a relatively high loading stiffness before it starts to curl over reaching a quasi-plastic state and eventual failure.
3.2 EFFECTS OF TOX JOINTS ON STEEL PERFORMANCE

3.2.1 General

The objective of this test series was to investigate the influence of TOX joints on the performance of the base panel material. High-strength material, viz. G550 sheet steel, was the preferred material for the principal tension member of the hybrid steel deck in composite action since it would remove the need for large quantities of conventional reinforcement. Using material with a large tensile capacity would also reduce the thickness of the tension member compared to material with a lower strength which, in return, would bring down the overall weight of the steel deck. Typically, a process called cold reduction is used to increase the strength and hardness of sheet steels which, on the other hand, can severely reduce the ductility of the steel. Ductility is the ability of material to undergo sizeable plastic deformation without fracture and is of vital importance for steel used in a structural member to compensate for stress concentrations or further cold-working. The presence of press-joints in the base panel could exhibit such a stress concentration and result in a sudden fracture of the whole sheet once the yield stress is reached in the steel at the press-joint. To ensure adequate performance of thin steel members, ductility criteria based on an investigation of sheet steels (Dhalla and Winter, 1974a; Dhalla and Winter, 1974b) were included in the Australian/New Zealand and North American Design Standards. However, the investigation did not include high strength G550 sheet steels which resulted in a limitation of its design stress and/or applications allowed by the Standards. To overcome the lack of knowledge, a study on the ductility of G550 sheet steel was undertaken at the University of Sydney, Australia (Rogers and Hancock, 1996). This found that the ductility was dependent on the rolling direction of the sheet and did not meet the Dhalla and Winter requirements except for uniform elongation in the longitudinal direction. The study also included perforated tensile coupons and showed their ability to develop the full net section capacity under tensile load despite their low values of uniform elongation. Over the following years, further research on the stability and ductility of G550 steel members and connections was carried out (Rogers et al., 2003), leading to a change of the current Australian/New Zealand Design Standard (AS/NZS 4600, 2005), which now allows 90 % (instead of 75 %) of the specified values of yield stress and tensile strength to be used as the design stress for thicknesses between 0.6 and 0.9 mm of G550 sheet steel.

In the following study, 48 tensile coupons with three different widths (20, 30 and 40 mm) and two clinching directions (version A and B) were tested to investigate the influence of round press-joints on the performance of sheet steel and to determine how much the extruded material of the joint contributes to the ultimate strength of the coupons. The press-joints (the TOX-system was used) were 8 mm in diameter and connected by a small complementary sheet (20x60 mm) to the various tensile coupons. For the high-strength G550 material, three different thicknesses (0.55, 0.75 and 1.00 mm) were intended to be investigated in this series but the 0.55 mm material was not available at the time of testing. To compare the results with a more ductile material, tests on G300 sheet steel with two thicknesses (0.55 and 0.85 mm) were performed. In addition, ten solid specimens were included to
determine the basic material properties of the sheet steels, i.e. yield stress, ultimate strength and Young’s modulus.

### 3.2.2 Test setup

All tests were carried out under displacement control in an INSTRON universal testing machine (Model No. 6027), equipped with the standard 200 kN load cell and calibrated according to the NATA (National Association of Testing Authorities, Australia) specifications. Depending on the material type, different crosshead speeds were used to obtain a convenient strain rate. The same crosshead speed was used for specimens with or without press-joints. The tensile coupons were held on both sides with the standard grips of the testing machine and special care was taken to centre the specimen in the grips by plumbing it with respect to the vertical to reduce the possibility of load eccentricity. Two standard 50 mm gauge-length INSTRON clip-on extensometers (Model No. 2630-100) were attached directly to the specimen on opposite sides approximately 70 mm above the TOX centre point. These were mainly used as guide for material behaviour during testing and to determine approximate elastic strain rates. Load, crosshead movement and strains were recorded digitally using the software package MERLIN installed on a personal computer. An original gauge length of nominally 200 mm was scribed on the surface of each specimen to determine the percentage of overall elongation after fracture. A picture of the test set-up is given in Figure 3-12.

![Test set-up](image)

**Figure 3-12. Test set-up**

### 3.2.3 Specimen details

The size of the tensile coupons was chosen to accommodate all three different widths without changing the geometry of the coupons and was based on the Australian Standard methods for tensile testing of metals (AS 1391, 1991). There, dimensional requirements for test pieces of rectangular cross-section are given recommending a minimum parallel length of 220 mm for 40 mm wide sections and 210 mm for 20 mm wide sections. For both widths, a minimum transition radius of 20 mm is
suggested to ensure that fracture occurs in the central region of the test specimen, noting that a greater radius may be needed for material of low ductility, such as G550. Studies undertaken at the University of Sydney have shown that G550 sheet steels are sensitive to material imperfections and stress concentrations (Maladakis and Ayoub, 1994) which can affect their tensile behaviour. Tensile coupons with a sharp radius may fail prematurely at the gauge/radius junction due to stress concentrations caused by the sudden increase in cross-sectional area. In a subsequent study, specimens with a more gradual change in cross-sectional area, i.e. with a radius $r$ of 55 mm, were found to give good results (Rogers and Hancock, 1996). Therefore, the 55 mm transition radius was adopted for the tensile coupons of this test series. Final size and shape of the specimens can be seen in Figure 3-13.

Two different types of galvanized sheet steels (G550 and G300) were under investigation in this study, and specimens with a distinct material type and thickness were cut from the same coil of sheet steel. The 50x500 mm blanks were guillotined in the longitudinal (rolling) direction and shipped to BHP Steel Research Laboratories in Port Kembla, Australia to mill the tensile coupons to their final shape. A CNC milling machine was used to achieve accurate and consistent test specimens, in particular the required large radius of 55 mm. In a second stage, 20x60 mm blanks with a base metal thickness of 0.75 mm were guillotined from G300 sheet steel. The tensile coupons and the smaller blanks were then shipped to TOX SYSTEMS Pty. Ltd. in Melbourne, Australia to press-join the two pieces concentrically in the constant width gauge length of the test coupons, using a standard TOX ‘Powercylinder’ press. Two different versions of test specimens were manufactured to investigate the influence of punch direction. Version A was obtained by placing the small complementary sheet on the punch side, whereas the punch penetrates the tensile coupon first in version B. An illustration of both versions is given in Figure 3-14.
Each specimen was given a distinct alpha-numeric code, e.g. 550(100)30A-2, determined from basic information. The initial three numbers characterize the type of material used for the tensile coupon, e.g. G550, followed by a further three numbers in brackets which represent its nominal base metal thickness in millimetres times 100, e.g. 1.00 mm. The constant gauge width of the coupon (in mm) is stated in the subsequent two numbers, e.g. 30 mm, followed by a letter indicating the specimen type, i.e. solid (S), clinching direction (A) or clinching direction (B). For each coupon width of a particular material, at least two specimens were tested. The specimen number, e.g. 2 is therefore separated by a dash. The code does not include specifications of the small complementary sheets since they were all cut from the same 0.75 mm G300 sheet steel. All specimens in this series were connected with an 8 mm round joint made with the tool sets listed in Table 3-8.

Table 3-8. TOX manufacturing tool sets

<table>
<thead>
<tr>
<th>Material punch side</th>
<th>Material die side</th>
<th>Version</th>
<th>Punch No.</th>
<th>Die No.</th>
<th>Press force</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.55 mm (G300)</td>
<td>0.75 mm (G300)</td>
<td>A</td>
<td>A60100</td>
<td>BD8012</td>
<td>54 kN</td>
</tr>
<tr>
<td>0.75 mm (G300)</td>
<td>0.55 mm (G300)</td>
<td>B</td>
<td>A60100</td>
<td>BB8012</td>
<td>54 kN</td>
</tr>
<tr>
<td>0.75 mm (G300)</td>
<td>0.85 mm (G300)</td>
<td>A</td>
<td>A60100</td>
<td>BB8012</td>
<td>54 kN</td>
</tr>
<tr>
<td>0.75 mm (G300)</td>
<td>0.75 / 1.00 mm (G550)</td>
<td>B</td>
<td>A60100</td>
<td>BB8012</td>
<td>54 kN</td>
</tr>
</tbody>
</table>

Measuring the inside diameter caused by the imprint of the punch was necessary to obtain the reduced cross-sectional area of the tensile coupon. Depending on the complementary sheet arrangement, different measurements had to be taken. Since the punch penetrates the tensile coupon first in version B, the inside diameter $i_B$ was measured. In version A the complementary sheet is placed on the punch side, therefore the inside diameter $i_A$ was measured, as indicated in Figure 3-14. Average values for the inside diameter of the different materials is given in Table 3-9.

Table 3-9. Average inside diameter of TOX joint

<table>
<thead>
<tr>
<th>Material</th>
<th>Version A ($i_A$) (mm)</th>
<th>Version B ($i_B$) (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.55 mm (G300) / 0.75 mm (G300)</td>
<td>7.15</td>
<td>6.20</td>
</tr>
<tr>
<td>0.85 mm (G300) / 0.75 mm (G300)</td>
<td>6.82</td>
<td>6.20</td>
</tr>
<tr>
<td>0.75 mm (G550) / 0.75 mm (G300)</td>
<td>6.83</td>
<td>6.20</td>
</tr>
<tr>
<td>1.00 mm (G550) / 0.75 mm (G300)</td>
<td>6.70</td>
<td>6.20</td>
</tr>
</tbody>
</table>

3.2.4 Material properties

Basic mechanical properties, i.e. yield stress, ultimate strength and Young’s modulus of the sheet steels used in the test series, were obtained through tensile testing of the solid specimens, viz. specimens without a press-joint. Since all specimens were cut in the longitudinal direction, material properties are only valid for this direction, noting that yield stress and ultimate strength of G550 steel are significantly higher for coupons obtained from transverse direction (Rogers and Hancock, 1996).

Tests for G550 sheet steels were run with a constant crosshead speed of 0.35 mm/min, which corresponds to an approximate elastic strain rate of 0.63E-3 mm/mm/min, whereas G300 sheet steels were initially tested at a crosshead speed of 1.8 mm/min (corresponds to an approximate elastic strain rate of 1.1E-3 mm/mm/min) and then slowly increased to 5.8 mm/min, once they reached about 7% elongation. Each specimen was first cycled three times to about 50% of its expected ultimate load before continuing until failure. As is common practice, material properties were calculated using base
metal thickness (b.m.t.) with data obtained from coupons tested in its original coated state. Individual test results for all solid specimens are included in Appendix C. The average results for the different materials used in this test series are listed in Table 3-10.

<table>
<thead>
<tr>
<th>Coupon</th>
<th>Coating</th>
<th>$t_c$ (mm)</th>
<th>$t_b$ (mm)</th>
<th>$f_y$ (MPa)</th>
<th>$\varepsilon_{0.2}$ (%)</th>
<th>$f_u$ (MPa)</th>
<th>$E^*$ (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>G300/0.85 coated</td>
<td>0.92</td>
<td>0.86</td>
<td>330.6</td>
<td>0.37</td>
<td>378.3</td>
<td>226.3</td>
<td></td>
</tr>
<tr>
<td>G300/0.55 coated</td>
<td>0.60</td>
<td>0.55</td>
<td>365.4</td>
<td>0.39</td>
<td>397.4</td>
<td>221.7</td>
<td></td>
</tr>
<tr>
<td>G550/1.00 coated</td>
<td>1.06</td>
<td>0.99</td>
<td>624.4</td>
<td>0.51</td>
<td>635.7</td>
<td>222.7</td>
<td></td>
</tr>
<tr>
<td>G550/0.75 coated</td>
<td>0.84</td>
<td>0.76</td>
<td>714.0</td>
<td>0.52</td>
<td>716.2</td>
<td>229.1</td>
<td></td>
</tr>
</tbody>
</table>

* values based on b.m.t.

### 3.2.5 Test results

Typical load-displacement curves for the G300 material are given in Figure 3-15 and for the G550 material in Figure 3-16. For each material thickness, an example of version A and of version B is presented. The shape of the curves are typical for the material tested showing gradual yielding with minimal strain hardening for the high-tensile G550 material, and a distinct yield plateau followed by strain hardening for the G300 material. Higher ultimate loads and greater displacement values were recorded for wider tensile coupons.

Typically, fracture of the material inside the TOX joint initiated failure for both versions of the 0.55 mm G300 material. The initiation of failure is quite noticeable as a sudden drop of load with the load continuing to decrease while the fracture opens up from the inside and necking of the ductile material occurs from the outside. Version A of the 0.85 mm G300 material first experienced a slow reduction of load before the load drops rapidly. In this version, the complementary sheet is located on the punch side, leaving a greater inside diameter on the coupon and minor necking occurs prior to fracture of the material inside the press-joint. In contrast, the complementary sheet in version B prevents fracture of the material inside the press-joint by providing a restraint. The fracture occurs around the neck of the joint accompanied with necking of the coupon.

The behaviour of the less ductile G550 material is quite different to the G300 material and more sensitive to cross-sectional changes. In general, no necking of the coupons and a rather brittle fracture could be observed. Independent of the material thickness, coupons of version A experienced a sharp drop of load shortly after reaching ultimate load which was usually lower compared to version B due to the greater inside diameter of the press-joint. The high tensile forces deformed the joint on the coupon longitudinally, causing the complementary sheet to be propelled out of its clinch. At the same time, fracture started from the inside of the joint and travelled instantly across the whole width of the coupon. In version B the complementary sheet is placed on the die side, forcing the fracture to occur around the neck of the joint. Prior to fracture, the load decreased slowly as soon as the ultimate load was reached. Various failure patterns were observed for the tensile coupon specimens tested, and photographs of typical examples for each material are displayed in Figure 3-17.
Figure 3-15. Typical load-displacement curves for G300 material

Figure 3-16. Typical load-displacement curves for G550 material
The coupon failure patterns could be classified into five different fracture modes, viz. transverse (mode 1), diagonal 1 (mode 2), diagonal 2 (mode 3), transverse/diagonal 1 (mode 4) and transverse/diagonal 2 (mode 5). The fracture modes are shown in Figure 3-18 and listed in Table 3-11 for the G300 material and in Table 3-12 for the G550 material. On some occasions, specimens failed
Chapter 3. Basic behaviour of TOX joints

by a combination of failure modes, e.g. transverse/diagonal 1 on one side and diagonal 1 on the other (mode 4+2). All solid specimens failed in failure mode 2.

Figure 3-18. Coupon Failure Patterns

Table 3-11. Elongation and failure modes of G300 material

<table>
<thead>
<tr>
<th>Code</th>
<th>$\varepsilon_{200}$ (%)</th>
<th>Fracture mode</th>
<th>Code</th>
<th>$\varepsilon_{200}$ (%)</th>
<th>Fracture mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>300(055)S-1</td>
<td>27.68</td>
<td>2</td>
<td>300(085)S-1</td>
<td>28.73</td>
<td>2</td>
</tr>
<tr>
<td>300(055)S-2</td>
<td>29.14</td>
<td>2</td>
<td>300(085)S-2</td>
<td>28.50</td>
<td>2</td>
</tr>
<tr>
<td>300(055)S-3</td>
<td>27.57</td>
<td>2</td>
<td>300(085)S-3</td>
<td></td>
<td></td>
</tr>
<tr>
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</tr>
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<td>1+4</td>
<td>300(085)40B-1</td>
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<td>1</td>
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Table 3-12. Elongation and failure modes of G550 material

<table>
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<th>Code</th>
<th>$\varepsilon_{200}$ (%)</th>
<th>Fracture mode</th>
<th>Code</th>
<th>$\varepsilon_{200}$ (%)</th>
<th>Fracture mode</th>
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<td>550(100)S-1</td>
<td>5.55</td>
<td>2</td>
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<tr>
<td>550(075)S-2</td>
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<td>550(100)S-2</td>
<td>5.64</td>
<td>2</td>
</tr>
<tr>
<td>550(075)S-3</td>
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<td></td>
<td>550(100)S-3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>550(075)20A-1</td>
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<td>550(100)20A-1</td>
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<td>550(075)40A-2</td>
<td>1.09</td>
<td>4</td>
<td>550(100)40A-2</td>
<td>1.45</td>
<td>4</td>
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<td>550(100)20B-1</td>
<td>1.09</td>
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</tr>
<tr>
<td>550(075)20B-2</td>
<td>1.05</td>
<td>2+4</td>
<td>550(100)20B-2</td>
<td>0.91</td>
<td>2+4</td>
</tr>
<tr>
<td>550(075)30B-1</td>
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<td>2+4</td>
<td>550(100)30B-1</td>
<td>1.00</td>
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<tr>
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<td>0.73</td>
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<td>550(100)30B-2</td>
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<td>4</td>
</tr>
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<td>4</td>
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</tbody>
</table>

The percentage elongation after fracture on a 200 mm gauge length $\varepsilon_{200}$ for both thicknesses of the G300 material was around 28 % for solid specimens. The presence of press-joints reduces the ability of the sheet to deform considerably and is more prominent for narrower coupons, with a percentage elongation of approximately 14 % for the 20 mm wide coupons of both thicknesses. Deformations of
specimens made out of the high-strength G550 material were significantly smaller. The percentage elongation after fracture for a 0.75 mm and 1.00 mm thick solid specimen measured only about 1.3 % and 5.6 % respectively. The press-joints reduce the percentage elongation to values in the order of approximately 1 % for the 0.75 mm material, and slightly higher values for the 1.00 mm material. Individual results can be found in Table 3-11 and Table 3-12.

Since press clinching does not actually remove material from the gross cross-sectional area, the term net cross-sectional area is not appropriate and the ratio \( \frac{il}{b} \) is introduced instead. This ratio describes the influence of the press-joint in relation to the width \( b \) of the tensile coupon. Depending on the material thickness and on which side the complementary sheet is located, the inside diameter \( i \) of the press-joint varies although the same punch tool was used for all materials tested in this series. The ratio ranges from 0.15 up to 0.36 for coupons with round press-joints, and is obviously zero for solid specimens. For each specimen, the ultimate load reached during testing \( P_u \), the load at initiation of fracture \( P_i \) and the load at final fracture \( P_f \) were recorded. Table 3-13 to Table 3-16 list the results for all tested specimens, including for the solid coupons. The ratio \( \frac{P_i}{P_u} \) represents a measure of the local straining which occurs after the ultimate load was reached to the point of initiation of fracture at the press-joint, whereas the ratio \( \frac{P_f}{P_u} \) characterizes the total local straining which occurs after ultimate load to the point of final fracture.

To make a comparison between the different sized specimens possible, the ultimate stress capacity \( f_u \) was calculated by two methods: (a) by using the gross cross-sectional area \( A_{sb} \) and assuming the press-joint does not reduce the cross-section at all; and (b) by using the net cross-sectional area \( A_{hb} \) and assuming the press-joint reduces the cross-section as much as an actual hole. The area was based on the base metal thickness \( t_b \) of the material and following equations were used to calculate the ultimate stress capacities:

\[
\begin{align*}
(a) \quad f_{us} &= \frac{P_u}{b t_b} \\
(b) \quad f_{uh} &= \frac{P_u}{(b-i) t_b}
\end{align*}
\] (3-7) (3-8)

The ultimate stress capacities of each specimen with a press-joint were then divided by the corresponding average ultimate stress capacity \( f_{u,m} \) of the actual corresponding solid specimens.

The ratio \( f_{us}/f_{u,m} \) gives an upper bound on how much the extruded material of the press-joint contributes to the ultimate strength of the material, whereas the ratio \( f_{uh}/f_{u,m} \) gives a lower bound on how much the press-joint has reduced the strength of the solid gross-section. A ratio of 1.0 indicates that the stress in the test specimen, calculated on the basis of either an assumed solid section or a fully holed section, has reached the ultimate tensile strength of the material. This is shown in Figure 3-19 for the G300 material and in Figure 3-20 for the G550 material.
Table 3-13. Test results of G300 material, \( t=0.55 \) mm

<table>
<thead>
<tr>
<th>Code</th>
<th>( \frac{i}{b} )</th>
<th>( P_u ) (kN)</th>
<th>( \frac{P_i}{P_u} ) (-)</th>
<th>( \frac{P}{P_u} ) (-)</th>
<th>( f_{us} ) (MPa)</th>
<th>( \frac{f_{us}}{f_{u,m}} ) (-)</th>
<th>( f_{uh} ) (MPa)</th>
<th>( \frac{f_{uh}}{f_{u,m}} ) (-)</th>
</tr>
</thead>
<tbody>
<tr>
<td>300(055)S-1</td>
<td>0.00</td>
<td>4.37</td>
<td>n/a</td>
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<td>395.0</td>
<td>0.99</td>
<td>n/a</td>
<td>n/a</td>
</tr>
<tr>
<td>300(055)S-2</td>
<td>0.00</td>
<td>4.40</td>
<td>n/a</td>
<td>0.90</td>
<td>398.2</td>
<td>1.00</td>
<td>n/a</td>
<td>n/a</td>
</tr>
<tr>
<td>300(055)S-3</td>
<td>0.00</td>
<td>4.40</td>
<td>n/a</td>
<td>0.90</td>
<td>398.8</td>
<td>1.00</td>
<td>n/a</td>
<td>n/a</td>
</tr>
<tr>
<td>Average</td>
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<td>0.90</td>
<td>397.3</td>
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Table 3-14. Test results of G300 material, \( t=0.85 \) mm

<table>
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<tr>
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<th>( P_u ) (kN)</th>
<th>( \frac{P_i}{P_u} ) (-)</th>
<th>( \frac{P}{P_u} ) (-)</th>
<th>( f_{us} ) (MPa)</th>
<th>( \frac{f_{us}}{f_{u,m}} ) (-)</th>
<th>( f_{uh} ) (MPa)</th>
<th>( \frac{f_{uh}}{f_{u,m}} ) (-)</th>
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<tbody>
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<td>300(085)S-1</td>
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<td>n/a</td>
<td>0.85</td>
<td>379.3</td>
<td>1.00</td>
<td>n/a</td>
<td>n/a</td>
</tr>
<tr>
<td>300(085)S-2</td>
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<td>6.50</td>
<td>n/a</td>
<td>0.85</td>
<td>377.3</td>
<td>1.00</td>
<td>n/a</td>
<td>n/a</td>
</tr>
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Figure 3-19. Effect of TOX on ultimate strength capacity of G300 material
Table 3.15. Test results of G550 material, t=0.75 mm

<table>
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<th>i/b</th>
<th>Pu  (kN)</th>
<th>Pi/Pu (-)</th>
<th>Pf/Pu (-)</th>
<th>f us (MPa)</th>
<th>f us/fu,m (-)</th>
<th>f hm (MPa)</th>
<th>f hm/fu,m (-)</th>
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</thead>
<tbody>
<tr>
<td>550(075)S-1</td>
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<td>10.95</td>
<td>n/a</td>
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<td>1.00</td>
<td>n/a</td>
<td>n/a</td>
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<td>n/a</td>
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<td>715.6</td>
<td>1.00</td>
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<td>n/a</td>
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<td><strong>0.75</strong></td>
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<td><strong>716.2</strong></td>
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<td>0.96</td>
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<td>568.2</td>
<td>0.79</td>
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<td>0.95</td>
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<td>0.78</td>
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<td>1.17</td>
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<td>550(075)30A-1</td>
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<td>0.96</td>
<td>0.16</td>
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<td>0.96</td>
<td>0.00</td>
<td>607.0</td>
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<td>19.28</td>
<td>0.98</td>
<td>0.13</td>
<td>630.9</td>
<td>0.86</td>
<td>760.0</td>
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<td>0.97</td>
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<td>635.3</td>
<td>0.89</td>
<td>765.7</td>
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<td>550(075)20B-1</td>
<td>0.31</td>
<td>9.50</td>
<td>0.89</td>
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<td>0.86</td>
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<td>650.3</td>
<td>0.91</td>
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<td>0.91</td>
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Table 3.16. Test results of G550 material, t=1.00 mm

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<th>Pi/Pu (-)</th>
<th>Pf/Pu (-)</th>
<th>f us (MPa)</th>
<th>f us/fu,m (-)</th>
<th>f hm (MPa)</th>
<th>f hm/fu,m (-)</th>
</tr>
</thead>
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<td>0.72</td>
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<td>0.92</td>
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<td>0.94</td>
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<td>617.4</td>
<td>0.97</td>
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<td>0.95</td>
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<td>598.8</td>
<td>0.94</td>
<td>718.9</td>
<td>1.13</td>
</tr>
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<td>0.16</td>
<td>563.6</td>
<td>0.89</td>
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<td>607.1</td>
<td>0.96</td>
<td>718.0</td>
<td>1.13</td>
</tr>
<tr>
<td>550(100)40B-2</td>
<td>0.15</td>
<td>24.31</td>
<td>0.80</td>
<td>0.08</td>
<td>610.9</td>
<td>0.96</td>
<td>722.3</td>
<td>1.14</td>
</tr>
</tbody>
</table>

Figure 3.20. Effect of TOX on ultimate strength capacity of G550 material
Chapter 3. Basic behaviour of TOX joints

From Figure 3-19 it becomes very obvious that round press-joints have a very small effect on the ultimate strength capacity of G300 material. The behaviour is more comparable to a solid section than a holed section and suggests that the extruded material of the press-joint is able to carry almost as much load as an undisturbed, solid piece. The results for the strengths calculated for the holed section show artificially high values of ultimate tensile strength (relative to the actual ultimate tensile strength of the specimen), indicating that the net-cross sectional area based on an assumed hole greatly overestimates the reduction in area provided by the press-joint. Independent of the material thicknesses and versions, the results show a similar linear relationship for the solid and holed specimens, suggesting that the influence of an 8 mm press-joint can be fully ignored, especially for $i/b$ ratios less than 0.15.

From Figure 3-20 it is evident that round press-joints have a considerable effect on the ultimate strength capacity of G550 material, particularly for the higher $i/b$ ratios where the actual tensile strength based on a solid section is less than the actual tensile strength of the specimen. This behaviour is less severe for specimens of version B due to the generally smaller inside diameter $i$. However, the results for the strengths calculated on the basis of a holed section still show artificially high values of ultimate tensile strength (relative to the actual ultimate tensile strength), indicating that the net-cross sectional area based on an assumed hole still overestimates the reduction in area provided by the press-joint, but not to the same extent as for the G300 material. This means that the extruded material of the press-joint can still carry some of the stress, but is not fully effective. It would be overly conservative to treat the press-joint as a hole of the same inside diameter $i$ as the joint itself, but it could be considered as a hole with a reduced diameter $i_{\text{red}}$ calculated using equation (3-8), where $f_{ub}$ is set to a value of the actual tensile strength of the specimen. For specimen 550(100)20A-1, the reduced inner diameter would be 3.19 mm compared to the original inner diameter of 6.70 mm. Other values of reduced diameter can be found in Appendix C. However, for practical sizes of the base panels of the hybrid steel deck, the ratios $i/b$ would be rather small, typically 0.05, in which case the reduction in strength due to the TOX could be ignored.

3.2.6 Conclusions

Earlier studies on the tensile behaviour of thin G550 sheet steel have shown that perforated specimens are able to develop the full net cross-section ultimate tensile strength. Tests on tensile coupons incorporating round TOX press-joints have now demonstrated that the extruded material of these joints is able to carry some of the stress and behave better than an actual hole with the same inside diameter as the joint. In the case of G300 material, the behaviour is comparable to a solid section whereas the actual tensile strength for G550 material is less than the actual tensile strength when based on a solid section. However, treating the press-joint as a hole of the same size would greatly overestimates the reduction in area provided by the press-joint for higher $i/b$ ratios. For very low $i/b$ ratios, the effect of the TOX on the ultimate tensile strength could be ignored.
4.1  GENERAL

The hybrid steel deck distinguishes itself quite substantially from other steel decks normally used in composite construction. Although the cross-sectional shape resembles a deep trapezoidal deck, the main difference can be found in the modular approach employed by the hybrid steel deck. Here, the section is formed by connecting individual components to each other whereas a single, flat steel sheet is used to form the shape of the trapezoidal deck. This difference results in many advantages for the hybrid steel deck (e.g. varying metal thicknesses for individual components; web openings allow the passage of reinforcing bars as well as the entire or local filling of the cellular section with concrete; panels can be precambered) compared to ordinary trapezoidal decks. Also the behaviour is fundamentally different and has not been studied before.

The preliminary tests were initially conducted as part of product verification for practical applications in commercial projects. The geometries were therefore dictated by the requirements of the particular project and were not directly part of a systematic research program. However, the testing of these trial sections did allow the development of an understanding of the flexural behaviour of the decking and vital knowledge about its overall performance and possible failure modes was gained. It also allowed the development of an appropriate testing apparatus and associated measurement devices.

Over 33 trial specimens have been tested. The trial sections varied from 90 mm to 140 mm in overall height and with spans from as short as 2.26 m up to a length of 8.46 m. In this range of sections, a variety of plate material combinations, connector spacing and web opening configurations were used. The thickness of the top plate ranged from 1.60 mm to 2.35 mm, the web from 0.70 mm to 0.90 mm and the base panel from 0.55 mm to 1.00 mm. Specimens with connectors spaced at 70 mm and 100 mm, as well as specimens with or without web openings, mostly spaced at 200 mm centres,
have been tested. On the basis of these tests, decisions were then made regarding the significant aspects of the behaviour that would be more closely investigated in the research of this thesis.

4.2 TEST SET-UP

A well-articulated loading frame has been built to conduct single span tests under simulated uniformly-distributed loading conditions with roller bearings as supports to allow horizontal movement of the specimen to occur freely during bending. A 200 mm wide and 5 mm thick plate was fitted over the roller supports to better distribute the reaction forces. Both ribs of the base panel were clamped to this plate, simulating lateral restraint from adjacent panels in an actual application. This was regarded necessary as base panels (in particular base panels of minor thickness) with unrestrained ribs tend to flatten under localized pressure over the supports. The applied jack load was diverted into four line loads of equal magnitude to simulate uniformly-distributed loading conditions. Two loading rigs were rigidly connected to a spreader beam which was attached to the loading beam via a pivot point. The jack load was applied through a 1000 kN capacity MTS hydraulic actuator with a 250 mm stroke that allowed the use of pre-programmed stroke rates. The rates were usually in the range of 1 to 2 mm/min during the loading process. The loading arrangement can be seen in Figure 4-1.

The design of the loading rig was influenced by a number of considerations. The primary requirement was to accommodate the distinct shape of the hybrid steel deck which can take on different heights from 90 mm (TD 90) up to possibly 210 mm (TD 210). Irrespective of the section height, the base panel has a constant nominal width of 250 mm. Various loading arrangement have been considered, and loading on the top of the bottom web flanges was chosen to be the most suitable for a truss-truss arrangement. The nominal 200 mm wide top plate of the TD 90/TD 110 sections, as well as the confined space between the corrugated web and the rib of the base panel, made direct loading impossible. Therefore, purpose-designed steel loading brackets were built which transmit the applied load equally onto the top side of the bottom web flanges on each side of the specimen. The stiff brackets passed underneath the panels with “hooks” at each end that passed up over the ribs. The top surface of the hooks formed a horizontal platform onto which the load could then be applied through a set of loading fingers. Small metal packers, approximately 25 mm long and 3 mm thick, were used in between the press-joints to firstly provide an even area for the loading brackets and secondly not to directly load the connectors which could possibly influence their structural

![Figure 4-1. Loading frame with four loading points – Side View](image-url)
Chapter 4. Preliminary tests on hybrid steel deck

performance. Roller bearings were incorporated at the end of each loading finger to minimise longitudinal frictional restraint. Each set of loading fingers was rigidly connected to a crossbeam which was then attached to the spreader beam. The loading brackets were 100 mm wide to avoid concentrated loading of the sheeting, as well as to allow the loading fingers to travel longitudinally during the loading process. The loading rig with a variety of possible cross-sections can be seen in Figure 4-2.

![Figure 4-2. Loading rig – Front view](image)

With expected loads below 50 kN, an external 100 kN load cell (Type TML-TCLM-10B) was used to measure the total applied jack load. The load cell was previously calibrated according to the NATA (National Association of Testing Authorities, Australia) specifications. Linear potentiometers (LP’s) of the type SAKAE-S18FLPA100R with a range of 100 mm were used to typically measure vertical deflection at mid-span and both quarter-spans at three distinctive points across the base panel (North, Centre, South). Deflection and load measurements were captured using a data acquisition system (DATA TAKER DT605, Series 3) and the software package DELOGGER PRO. Figure 4-3 illustrates the instrumentation of the test panel whereas Figure 4-4 shows the test set-up in operation.

![Figure 4-3. Instrumentation of test specimen](image)
4.3 POSITIVE BENDING TESTS

4.3.1 General

Positive bending tests of the hybrid steel deck give vital information about its overall behaviour and failure modes where the top plate is in compression and the base panel in tension. This is typical for the application of the hybrid steel deck in practice where it is simply supported on either steel beams or walls. During the course of these preliminary series, several improvements to the hybrid steel deck have been undertaken to increase its performance. The first modification was the extension of the length of the web corrugations, reducing the portion of flat web at the intersections to the upper and lower web flange and therefore reducing the possibility of web crippling in this region. The second improvement was the introduction of edge stiffeners to the initially unstiffened edge of the top plate. Soon after, the geometry of the longitudinal intermediate stiffeners was entirely changed. The last modification was the introduction of a new, improved end-diaphragm. The following illustration (Figure 4-5) shows the different geometries of the top plate, whereas Figure 4-6 gives examples of various diaphragm types.

![Figure 4-4. Test set-up (positive bending tests)](image)

![Figure 4-5. Geometries of various top plate (TP) types](image)
Table 4-1. Specimen details

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Section</th>
<th>L (m)</th>
<th>f_{TP} (mm)</th>
<th>f_{WB} (mm)</th>
<th>f_{BP} (mm)</th>
<th>TP-Type</th>
<th>Dia-Type</th>
<th>S_{holes} (mm)</th>
<th>S_{ROX} (mm)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>NRA-01</td>
<td>TD140</td>
<td>3.40</td>
<td>2.35</td>
<td>0.90</td>
<td>0.55</td>
<td>A</td>
<td>A</td>
<td>320</td>
<td>100</td>
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</tr>
<tr>
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<td>2.35</td>
<td>0.90</td>
<td>0.55</td>
<td>A</td>
<td>A</td>
<td>320</td>
<td>100</td>
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<tr>
<td>NRB-01</td>
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<td>0.90</td>
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<td>A</td>
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</tr>
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<td>0.90</td>
<td>0.55</td>
<td>A</td>
<td>A</td>
<td>-</td>
<td>100</td>
<td></td>
</tr>
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<td>2.35</td>
<td>0.90</td>
<td>0.55</td>
<td>A</td>
<td>A</td>
<td>-</td>
<td>100</td>
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</tr>
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<td>0.75</td>
<td>A</td>
<td>A</td>
<td>-</td>
<td>100</td>
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</tr>
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<td>0.90</td>
<td>1.00</td>
<td>A</td>
<td>A</td>
<td>-</td>
<td>100</td>
<td></td>
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<td>0.70</td>
<td>0.55</td>
<td>A</td>
<td>A</td>
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<td>100</td>
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<td>70</td>
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<td>70</td>
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</tr>
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<td>A</td>
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<td>100</td>
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<td>1.66</td>
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<td>0.75</td>
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<td>A</td>
<td>-</td>
<td>70</td>
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</tr>
<tr>
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<td>1.90</td>
<td>0.70</td>
<td>1.00</td>
<td>C</td>
<td>A</td>
<td>-</td>
<td>100</td>
<td>Plates at support</td>
</tr>
<tr>
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<td>TD90</td>
<td>2.40</td>
<td>1.90</td>
<td>0.70</td>
<td>1.00</td>
<td>C</td>
<td>A</td>
<td>-</td>
<td>70</td>
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<td>B</td>
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<td>A</td>
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<td>100</td>
<td>Plates at support</td>
</tr>
<tr>
<td>EXS-01</td>
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<td>5.18</td>
<td>2.40</td>
<td>0.70</td>
<td>0.75</td>
<td>C</td>
<td>A</td>
<td>205</td>
<td>100</td>
<td>Plates at support</td>
</tr>
<tr>
<td>EXS-02</td>
<td>TD110</td>
<td>5.18</td>
<td>2.40</td>
<td>0.90</td>
<td>0.75</td>
<td>C</td>
<td>A</td>
<td>205</td>
<td>100</td>
<td>Plates at support</td>
</tr>
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<td>1.00</td>
<td>C</td>
<td>B</td>
<td>200</td>
<td>100</td>
<td>Plates at support</td>
</tr>
<tr>
<td>WMC-02</td>
<td>TD140</td>
<td>2.26</td>
<td>1.88</td>
<td>0.70</td>
<td>1.00</td>
<td>C</td>
<td>C</td>
<td>200</td>
<td>100</td>
<td>Plates at support</td>
</tr>
<tr>
<td>WMC-03</td>
<td>TD140</td>
<td>2.26</td>
<td>1.88</td>
<td>0.70</td>
<td>1.00</td>
<td>C</td>
<td>D</td>
<td>200</td>
<td>100</td>
<td>t_{BA} = 0.7 mm</td>
</tr>
<tr>
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<td>2.26</td>
<td>1.88</td>
<td>0.70</td>
<td>1.00</td>
<td>C</td>
<td>D</td>
<td>200</td>
<td>100</td>
<td>t_{BA} = 1.0 mm</td>
</tr>
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<td>1.90</td>
<td>0.70</td>
<td>0.75</td>
<td>C</td>
<td>D</td>
<td>205</td>
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</tr>
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<td>C</td>
<td>D</td>
<td>195</td>
<td>100</td>
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<tr>
<td>TRI-01</td>
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<td>1.90</td>
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<td>0.75</td>
<td>C</td>
<td>D</td>
<td>200</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>TRI-02</td>
<td>TD110</td>
<td>4.72</td>
<td>1.90</td>
<td>0.90</td>
<td>0.75</td>
<td>C</td>
<td>D</td>
<td>200</td>
<td>100</td>
<td>2 holes near support</td>
</tr>
<tr>
<td>ANU-01</td>
<td>TD140</td>
<td>4.055</td>
<td>3.00</td>
<td>0.70</td>
<td>1.00</td>
<td>C</td>
<td>C</td>
<td>-</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>ANU-02</td>
<td>TD140</td>
<td>7.23</td>
<td>3.00</td>
<td>0.90</td>
<td>1.00</td>
<td>C</td>
<td>D</td>
<td>various</td>
<td>100</td>
<td>various double holes</td>
</tr>
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<td>8.46</td>
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<td>1.00</td>
<td>C</td>
<td>D</td>
<td>200</td>
<td>100</td>
<td>3 holes near support</td>
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<td>C</td>
<td>D</td>
<td>100</td>
<td>100</td>
<td></td>
</tr>
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<td>0.90</td>
<td>0.75</td>
<td>C</td>
<td>D</td>
<td>various</td>
<td>100</td>
<td>various double holes</td>
</tr>
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<td>D</td>
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<td>100</td>
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<td>RQY-01</td>
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<td>1.00</td>
<td>C</td>
<td>D</td>
<td>200</td>
<td>100</td>
<td></td>
</tr>
</tbody>
</table>

Table 4-1 gives an overview of the main details for the trial specimens tested in this preliminary series. The overall section height varied from 90 mm (TD90) to 140 mm (TD140) and specimens with single spans \( L \) from as short as 2.26 m up to a length of 8.46 m were tested. Nominal base material thicknesses are given for each component of the section, viz. the top plate \( f_{TP} \), the corrugated web \( f_{WB} \), and the base panel \( f_{BP} \). The base panel was typically made of high-tensile (G550) galvanized steel, whereas a lower grade steel (G350) was used for the webs. The grade of the top plate (TP) varied, but was usually either HA70T or Blackform steel. Specimens with or without web openings (90 x 40 mm) have been tested where the spacing from centre to centre of the web openings \( S_{holes} \) was mostly a constant of 200 mm. In some cases, the web openings were only present at distinct locations to allow
the section to be filled locally with concrete, e.g. near the supports. The spacing of the TOX connectors $s_{\text{TOX}}$ was typically 100 mm. Some of the specimens experienced local buckling of the corrugated web over the support due to the specific design of the initial end-diaphragm (Dia-type A). Until an alternative and practical end-diaphragm was developed, the webs over the supports were locally stiffened with thick metal plates, tightly fitted between the top plate and the base panel.

### 4.3.2 Test results

Having a section that is formed by connecting individual components to each other, a range of principal modes of failure can potentially be expected. If the moment capacity of the section is not governed by the strength of the longitudinal shear connection between the components, i.e. between the corrugated web and the top plate or the base panel, either the base panel yields and possibly fractures in tension or the top plate yields or buckles in compression. Apart from these failure modes, buckling of the web may occur, in particular at the end regions over the supports where the highest shear forces are located. However, if the strength of the longitudinal shear connection governs the moment capacity of the section, the section may experience a shear failure of the connectors between the web and the top plate or the base panel. The strength of the shear connection is dependent on the spacing (i.e. total number) and strength of the connectors. In the case of the TOX connectors, the strength is a function of the material properties and thicknesses of the individual sheets forming the joint (see Chapter 3.1.5). It was assumed that the connectors between the relatively thin base panel and the webs shear off well before the connectors between the usually thick top plate and the web, bearing in mind that equal numbers and spacing of connectors are made in the top and the bottom of the hybrid steel deck due to the equipment used for assembling the panels (see Chapter 2.4.2).

#### Table 4-2. Test results

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$M_u$ (kNm)</th>
<th>$\delta_u/L$</th>
<th>Failure mode</th>
<th>Specimen</th>
<th>$M_u$ (kNm)</th>
<th>$\delta_u/L$</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>NRA-01</td>
<td>5.99</td>
<td>1 / 304</td>
<td>BPSH</td>
<td>EXS-01</td>
<td>10.64</td>
<td>1 / 57</td>
<td>BPSH</td>
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<td>NRA-02</td>
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<td>EXS-02</td>
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<td>BPSH</td>
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<td>NRB-01</td>
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<td>BPSH</td>
<td>WMC-01</td>
<td>8.41</td>
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<td>WBBU/HLDT</td>
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<td>WBBU/BPSH</td>
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<td>1 / 192</td>
<td>TPBU</td>
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<td>NRB-05</td>
<td>14.96</td>
<td>1 / 111</td>
<td>TPBU</td>
<td>FRA-01</td>
<td>5.87</td>
<td>1 / 94</td>
<td>n/a</td>
</tr>
<tr>
<td>PEL-01</td>
<td>4.90</td>
<td>1 / 77</td>
<td>BPSH</td>
<td>FRA-02</td>
<td>8.48</td>
<td>1 / 64</td>
<td>n/a</td>
</tr>
<tr>
<td>PEL-02</td>
<td>4.53</td>
<td>1 / 96</td>
<td>BPSH</td>
<td>TRI-01</td>
<td>11.79</td>
<td>1 / 47</td>
<td>HLDT</td>
</tr>
<tr>
<td>CAU-01</td>
<td>7.18</td>
<td>1 / 78</td>
<td>TPBU</td>
<td>TRI-02</td>
<td>11.80</td>
<td>1 / 52</td>
<td>HLDT</td>
</tr>
<tr>
<td>CAB-01</td>
<td>5.55</td>
<td>1 / 100</td>
<td>TPBU</td>
<td>ANU-01</td>
<td>14.58</td>
<td>1 / 63</td>
<td>TPBU</td>
</tr>
<tr>
<td>EUK-01</td>
<td>5.43</td>
<td>1 / 84</td>
<td>TPBU</td>
<td>ANU-02</td>
<td>17.79</td>
<td>1 / 58</td>
<td>TPBU</td>
</tr>
<tr>
<td>MIA-01</td>
<td>4.63</td>
<td>1 / 130</td>
<td>TPBU</td>
<td>ANU-03</td>
<td>16.21</td>
<td>1 / 53</td>
<td>TPBU</td>
</tr>
<tr>
<td>MIT-01</td>
<td>7.63</td>
<td>1 / 77</td>
<td>BPSH</td>
<td>ELT-01</td>
<td>9.94</td>
<td>1 / 40</td>
<td>TPBU</td>
</tr>
<tr>
<td>MIT-02</td>
<td>8.38</td>
<td>1 / 74</td>
<td>WBBU/BPSH</td>
<td>CAU-01</td>
<td>13.09</td>
<td>1 / 62</td>
<td>TPBU</td>
</tr>
<tr>
<td>MIT-03</td>
<td>6.99</td>
<td>1 / 67</td>
<td>TPBU</td>
<td>COL-01</td>
<td>16.95</td>
<td>1 / 35</td>
<td>TPBU</td>
</tr>
<tr>
<td>EWA-01</td>
<td>5.83</td>
<td>1 / 52</td>
<td>HLDT</td>
<td>RQY-01</td>
<td>12.53</td>
<td>1 / 40</td>
<td>TPBU</td>
</tr>
</tbody>
</table>

* Tests performed by UWS staff

A summary of the results of all specimens is given in Table 4-2. The table lists the ultimate moment $M_u$ applied to the specimen at midspan, as well as the corresponding midspan deflection $\delta_u$. The deflection is given as the ratio $\delta_u/L$ which is an indicator of the ductility of the specimen. A low value of the ratio indicates a specimen of low ductility. A reasonable limit for the ductility at the
ultimate limit state can be taken as 1/100 (Patrick and Bridge, 1988) as at this value the structure would show severe and perceivable deflections and obvious distress (Galambos and Ellingwood, 1986). For each specimen, the observed failure mode is also given.

In the following, some important findings that could be concluded from the conduct and observation of the tests for practical applications in commercial projects are presented. Figure 4-7 shows examples of some long spanning test specimens.

![Specimen ANU-03](image1)

![Specimen COL-01](image2)

![Specimen RQY-01](image3)

**Figure 4-7. Examples of some long spanning test specimens**

*Influence of base panel thickness*

The influence of the base panel thickness could be studied in a series of tests conducted on similar specimens without web openings that only differed in the nominal thickness of the base panels, viz. 0.55 mm (NRB-03), 0.75 mm (NRB-04) and 1.00 mm (NRB-05). By increasing the base panel thickness, the strength of the cellular cross-section can be increased. There are two obvious reasons. Changing the base panel thickness results in a shift of the neutral axis of the critical cross-section and consequently a change in the elastic section modulus and, depending on the height of the cross-section and the material and thicknesses of each component, either the compressive stresses in the top plate or the tensile stresses in the base panel may govern. For example, increasing the thickness of the base panel lowers the neutral axis and results in higher compressive stresses in the top plate relative to the base panel. This could lead to a failure of the top plate if the longitudinal shear connection in the base panel is strong enough and does not fail. Therefore, changing the stress distribution over the depth of a cross-section can have an effect on the principal failure mode. In addition, the thickness of the punch
side material, in this case the base panel thickness, determines the shear strength of the TOX connectors (see Chapter 3.1.5).

It can be seen from Table 4-2 that increasing the base panel thickness and consequently the shear strength of the TOX connectors also increased the ultimate moment $M_u$. Specimen NRB-03 and NRB-04 both experienced a longitudinal shear failure of the TOX connectors in the base panel (BPSH) near one end support as can be seen on the left in Figure 4-8 noting that the web and the base panel are purposely offset sideways to better expose the failure mode. This type of shear failure is similar to failure mode I observed previously when testing individual TOX connectors in shear (see Chapter 3.1.5). A further increase of base panel thickness (NRB-05) resulted in bottom TOX connectors that did not fail due to the applied moment. Instead, a different failure mode occurred where the top plate buckled under compression (TPBU). This buckle caused the upper web flanges and the top plate to separate with the connectors eventually failing locally by pull-out as a result of the increasing tensile forces, as can be seen on the right in Figure 4-8. It should also be noted that the buckling wavelength observed in the test was considerably larger than the spacing between the press-joints. This may be due to the stiffening effect of the two longitudinal stiffeners in the top plate.

Influence of connector spacing

In two comparable tests, the influence of different connector spacings, viz. 100 mm (MIT-01) and 70 mm (MIT-02) could be studied. Both specimens failed by longitudinal shearing of the connectors in the base panel (BPSH). The specimen with the closer-spaced connectors reached a 10% higher ultimate moment compared to the other specimen. Obviously, the number of shear connectors has a direct influence on how much longitudinal shear (and hence applied moment) can be transferred before the connection fails, provided that the top plate does not reach its yield or buckling capacity, and the shear strength of the bottom connectors governs failure. Although the influence of connector spacing, where top plate buckling governs, could not be directly studied during the testing of the trial sections, it is assumed that closer connector spacing could possibly increase the section moment capacity. This assumption is based on the premise that more connectors could increase the buckling strength of the top plate whereby the upper web flanges provide additional restraint, and could also increase the capability to sustain the subsequent tensile forces that develop between the two plate elements.
Influence of improved longitudinal stiffeners in the top plate

Based on the experience gained from the initial tests, it was decided to re-design the geometry of the top plate for better performance. Increasing the buckling capacity of the top plate would certainly enhance the efficiency of the cellular cross-section when buckling of the top plate governs. In extending the flat strip width of top plate type B by only 14% by increasing the depth and width of the two longitudinal stiffeners, the second moment of area of top plate type C could be increased by over 350%, whereas its centroid only shifted down by less than 1 mm. This improvement can be seen by comparing the results of specimen MIA-01 (plate type B) with specimen MIT-02 (plate type C). Both specimens were of similar geometry with no web openings and had a comparable span. Although the thicknesses of the top plates and the base panels were slightly different, a direct comparison of both specimens is still possible due to the experienced failure modes. In the tests, specimen MIT-02 was able to reach a 79% higher ultimate moment capacity compared to specimen MIA-01. Increasing the buckling capacity of the top plate not only had a positive effect on the strength of the cellular cross-section but also changed the mode of failure. Specimen MIA-01 experienced a distortional buckling of the top plate at ultimate load, with a subsequent failure of the connectors in the mid-span region by pull-out (TPBU). This was due to the lower buckling capacity of top plate type B in conjunction with the TOX connectors in the base panel that did not fail due to the applied moment. Even increasing the base panel thickness to 1.00 mm (as in specimen MIT-02) would not have changed the outcome of this test significantly since the buckling of the top plate still governed. In contrast, specimen MIT-02 with the improved longitudinal stiffeners in the top plate did not fail due to top plate buckling. Instead, local buckling of the corrugated webs over the end supports started to occur at a moment approximately 54% higher compared to that for specimen MIA-01. The higher moment resulted in higher end shear forces, causing the webs to buckle. To achieve the final ultimate moment $M_u$, the webs over the supports were locally stiffened with thick metal plates, tightly fitted between top plate and base panel (alternatively, the design of the end-diaphragms and the method of fixing the webs to them could have been improved, which it has subsequently and will be explained later in Chapter 4.4). When the ultimate moment was reached, the longitudinal shear connectors in the base panel experienced a shear failure while the top plate sustained the compressive forces without buckling.

Influence of the existence of web openings

In two comparable tests, the influence of web openings could be studied. The specimens had otherwise identical specifications and differed only by the presence of standard regularly-spaced, non-staggered web openings (90 mm long and 40 mm deep), located at mid-height of the webs. The spacing was 290 mm from centre to centre with the end regions slightly larger. A shear failure of the longitudinal shear connectors in the base panel (BPSH) occurred when the specimen without web openings (Specimen MIT-02) reached its ultimate moment, noting that the webs over the supports were locally stiffened with thick metal plates. In contrast, the specimen with web openings (Specimen MIT-03) reached an ultimate moment that was 20% lower compared to specimen MIT-02 and failed locally by pulling-out the connectors in the top plate and separation of the upper web flanges and the top plate (TPBU). The existence of web openings also increased the mid-span deflection at service
load, represented by a 50 mm concrete cover over the top plate, from $\delta/L = 1/600$ for specimen MIT-02 to $\delta/L = 1/400$ for specimen MIT-03. It is considered that the presence of openings in the web gives rise to the development of additional secondary bending moments at the top and bottom T-sections of the web due to Vierendeel-type deformation (Tse and Dayawansa, 1992). Failure is thought to be initiated by web distortion and local amplification of the tensile forces between the upper web flanges and the top plate. Figure 4-9 shows the distortion of the web openings (HLDT) near the support in a region of high shear and at midspan where the top plate buckled under compression.

![Figure 4-9. Distortion of web openings at midspan (left) and near support (right)](image)

**Influence of various end-diaphragm types**

Some of the specimens experienced local buckling of the corrugated web (WBBU) over the support due to the specific design of the initial end-diaphragm (type A), as can be seen on the left in Figure 4-10. In search of an alternative and practical end-diaphragm, a small series of tests was conducted (WMC-01 to 04). By re-designing the shape of the diaphragm, considerations of an open form was taken into account which would allow concrete to freely flow into the end regions of the panel when a solid slab is required for these regions. They would also allow hand access down the panels for installing services such as lighting. The final result was an open bracket-shaped diaphragm with a single internal stiffener that was simple to manufacture (Figure 4-6, type D). The standard end-diaphragm has an overall height of 75 mm and makes it particularly suited to the shallowest hybrid steel deck with a height of 90 mm. Using 0.7 mm thick material for the end-diaphragm (WMC-03) did not provide sufficient rigidity and resulted in a local web buckling failure over the support, as seen in the middle of Figure 4-10. Consequently, the material thickness was increased to 1.0 mm (WMC-04) which resulted in a quite different failure mode where the top plate moves laterally to one side with the sway being greatest at the ends. This lateral-distortional sway of the top plate (TPSW) can be seen on the right in Figure 4-10.

![Figure 4-10. Various failure modes: WBBU (left, centre) and TPSW (right)](image)
As already seen in this limited series of tests, the shapes and dimensions of the end-diaphragm can have a considerable influence on the failure behaviour of the hybrid steel deck. As a result, a numerical investigation into the effects that the open end-diaphragm would have on the local and global buckling of the cold-formed hybrid cellular deck was undertaken.

4.4 THE EFFECT OF END-DIAPHRAGMS

4.4.1 General

A finite element model, using the software package ABAQUS (ABAQUS, 2003), has been developed to simulate the effect of end-diaphragms on the hybrid steel deck. Similar to the positive bending tests described in Chapter 4.3, the model was established as a simply-supported span with a symmetric four-point loading arrangement. Three standard section heights, viz. 110 mm (TD 110), 140 mm (TD 140) and 160 mm (TD 160) have been investigated using the standard open end-diaphragm with an overall height of 75 mm. The influence of various diaphragm heights on a 160 mm high panel has also been examined. The idealized geometry and material thicknesses were based on the specifications given for specimen WMC-04 (see Table 4-1), but without web openings. In a parametric study, the span of the panels was varied from 1.5 m to 5.0 m. The analysis was performed as a linear-elastic eigenmode extraction (using the Lanczos eigensolver), initially assuming stress-free conditions without imperfections.

4.4.2 Finite element model

The finite element model (see Figure 4-11) represented the full geometry of the hybrid steel deck, thus ignoring any existing axes of symmetry. All panels extended past the support centrelines by 20 mm, the least that would occur in practice. The base sheet was supported in the vertical direction with the centre points restrained laterally. The symmetric four-point load was introduced via stiff plates (5 mm thick and 100 mm wide to match the width of the loading brackets in the actual tests) supported locally on the base sheet at $L/8$, $L/4$, $L/4$ and $L/8$ intervals. Four-node, doubly-curved thin shell elements with reduced integration, hourglass control and finite membrane strains (S4R-elements) were primarily used to model the cellular panel with suitable mesh refinements in the end support regions. The finite element mesh of the model was generated using the structured meshing technique and mainly quadrangular shaped elements. The initial 200 mm of the panel accommodated twenty elements in longitudinal direction, followed by a transition zone of 100 mm with five elements. Typically, a much coarser mesh was used for the section between the transition zones where less local deformation of the web was expected. For the present investigation, linear elastic material behaviour was considered with a modulus of elasticity $E$ of 200 GPa and a Poisson’s ratio $\nu$ of 0.3. The thicknesses of the top plate and the base sheet were 1.9 mm and 1.0 mm, respectively.
The 0.7 mm thick corrugated webs were modelled as flat plates of uniform thickness with orthotropic properties that model the effect of the corrugations. This modelling alternative has been used in many finite element analyses to simplify the geometry and therefore reduce analysis time without affecting accuracy as confirmed experimentally (Miller and Springer, 1979). An increased uniform thickness of 1.99 mm was used to match the flexural stiffness of the corrugated webs. In the longitudinal direction, a reduced elastic modulus, in conjunction with the uniform thickness, was used to match the reduced axial stiffness of the corrugated web. The reduced axial stiffness of the corrugations was obtained from a separate linear-elastic analysis of a longitudinal strip of the corrugations. Using these results, the elastic engineering constants (associated with the material’s principal directions: 1 in transverse, 2 in vertical and 3 in longitudinal direction) for the orthotropic properties were calculated for the corrugations. These can be found in Table 4-3 together with the values for the transition zone (1.4 mm thick) that was established at the end of the corrugations to model the diminishing corrugation amplitude.

### Table 4-3. Engineering constants for orthotropic plate properties

<table>
<thead>
<tr>
<th></th>
<th>$E_1$ (MPa)</th>
<th>$E_2$ (MPa)</th>
<th>$E_3$ (MPa)</th>
<th>$G_{12}$ (MPa)</th>
<th>$G_{13}$ (MPa)</th>
<th>$G_{23}$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Corrugation</td>
<td>200,000</td>
<td>200,000</td>
<td>24,700</td>
<td>76,900</td>
<td>20,700</td>
<td>20,700</td>
</tr>
<tr>
<td>Transition</td>
<td>200,000</td>
<td>200,000</td>
<td>35,100</td>
<td>76,900</td>
<td>19,800</td>
<td>19,800</td>
</tr>
</tbody>
</table>

The 1.0 mm thick stiffened end-diaphragm was modelled as a uniform thick plate with three different regions of stiffness equivalent to those of the actual diaphragm. The thickness at the base of the diaphragm was therefore 5.2 mm and reduced to 3.1 mm at a height of 30 mm. The remaining flat end of 15 mm was kept at the initial thickness of 1.0 mm. A plate thickness of 2.7 mm was used to model the smaller stiffener in the middle of the diaphragm. The end-diaphragm is locally connected to the base panel and the corrugated webs via self-piercing rivets (HENROB) which were modelled
using the *TIE-keyword. The connection points were located at heights of 30 mm and 60 mm above
the panel soffit, and in the middle of each longitudinal stiffener of the base sheet, as illustrated in
Figure 4-11.

The clinched connection between the upper web flanges and the top plate was modelled with
0.7 mm thick shell elements running continuously along the length of the panel. This simplification
was considered necessary in order to be able to perform an eigenvalue analysis because discrete
connection points would allow buckling of the upper web flanges through the top plate (not physically
possible) and consequently local buckling of the corrugated web. This simplification provides an
upper bound to the buckling values, assuming the clinched connection points are strong enough to
resist the buckling stress and are spaced sufficiently close, as is normal in practice, to prevent buckling
of the top plate in-between the connection points. As an eigenvalue analysis does not consider the slip
behaviour of the TOX connectors between the lower web flanges and the base panel, the web was also
continuously connected to the base panel.

4.4.3 Buckling modes

A number of different local and global practical buckling modes were detected for the range of
panels investigated. Overall views of the displaced shapes, and the displaced cross-sections at selected
points along the panel for the individual eigenmodes, are given in Figure 4-12 to Figure 4-15.

Figure 4-12. Lateral-distortional sway mode
Figure 4-13. Local top plate buckling mode

Figure 4-14. Local web buckling mode
In the lateral-distortional sway mode (Figure 4-12), the top plate moves laterally to one side, similar to what was observed for specimen WMC-04 in the preliminary test series (see Figure 4-10 on the right). This phenomenon is the equivalent to a failure mode (eigenmode 1) that exists for very short steel I-beams with an unrestrained top flange under flexure (Pircher et al., 2003). In the case of the hybrid steel deck, the end-diaphragm offers a restraint which significantly shortens the unsupported height of the web over the support region. However, the limited height of the end-diaphragm only restrains the lower part of the web, allowing lateral buckling of the upper part of the web with the top plate moving sideways.

For the local top plate buckling mode (Figure 4-13), the top plate reaches its elastic buckling capacity and forms buckles in the mid-span region where the bending moment is maximum. Although the numerical model did not account for individual connectors in the top plate, this type of failure occurs when the TOX joints between the upper web flanges and the top plate are closely-enough spaced to prevent buckling of the top plate in-between the connection points. This is normally the case with the connections 100 mm apart, as can be seen on the right in Figure 4-8. The buckling wavelength predicted by the model was similar to that observed in some experimental tests (around 400 mm) which is a function of the top plate thickness and other factors.

A local web buckling mode (Figure 4-14) only occurred with the larger section heights (TD 140 and TD 160) when the unsupported web above the end-diaphragms buckled over the support region. This failure mode was always secondary and required a higher critical buckling stress $f_{cr}$ than the lateral-distortional sway mode. In contrast, a similar web buckling mode was predicted to be the dominant failure mode for almost all the panels investigated when the end-diaphragm was completely...
left out (Figure 4-15). This, however, would never occur in sections for commercial projects where it is standard practice to fit an end-diaphragm over the support region.

It should be noted that the lateral-distortional sway mode and the local web buckling mode are mainly governed by the web and end-diaphragm configuration rather than the stiffness of the top plate whereas for the local top plate buckling mode, the stiffness of the top plate is the main variable that effects strength.

### 4.4.4 Results of linear-elastic eigenmode analysis

In Figure 4-16 to Figure 4-18, the ratio of the critical buckling stress $f_{cr}$ to the yield stress $f_y$ (which was set to 250 MPa) of the top plate associated with each eigenmode is plotted as a function of the span $L$ for three different section heights (TD 110, TD 140 and TD 160). It can be seen that lateral-distortional sway was the dominant failure mode for relatively short spans (up to about 2.0 m for TD 110 and 3.0 m for TD 140) and also for medium spans with a larger section height (4.0 m for TD 160), where the height of the unrestrained web above the diaphragm was greater, assuming the same standard end-diaphragm with the upper connection point located at a height of 60 mm was used. Further increasing the span resulted in a change of the dominant failure mode and top plate buckling started to govern. In none of the investigated cases, local web buckling was the dominant failure mode. However, if the end-diaphragm was left out, web buckling over the support was the dominant failure mode for almost all the panels investigated and was predicted to be 30% to 50% weaker than a panel with the end-diaphragm fitted that failed in a lateral-distortional sway mode. Only for spans over 3.5 m of the 110 mm high panel (TD 110) was top plate buckling predicted as the principal failure mode even without end-diaphragms.

It should be noted that the possibility of any failure (longitudinal shear or pull-out) regarding the TOX connectors in the base panel or in the top plate was not considered in this analysis. These types of failure would have to be assessed separately and could become the dominant failure mode.

### 4.4.5 Influence of the height of the end-diaphragm

Increasing the height of the end-diaphragm has a positive effect on the maximum critical buckling stress, particularly for the larger section heights. This was demonstrated for the 160 mm high panel (TD 160) where the height of the end-diaphragm was increased in 30 mm steps. The upper connection point was located at a height of 90 mm (with another two connection points at 30 and 60 mm), and also at a height of 120 mm (with another three connection points at 30, 60 and 90 mm). The thickness at the base of the 90 mm diaphragm was 5.2 mm and reduced to 3.1 mm at a height of 30 mm whereas the thickness of the 120 mm diaphragm was uniform (5.2 mm) up to a height of 120 mm. In addition, the 120 mm diaphragm had a stiffening plate with a reduced thickness of 3.1 mm located at the very top that prevented the sides to bend inwards, thus forming a box-shaped diaphragm similar to diaphragm type C (see Figure 4-6).
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Figure 4-16. Buckling modes for the 110 mm high panel (with standard end-diaphragm)

Figure 4-17. Buckling modes for the 140 mm high panel (with standard end-diaphragm)

Figure 4-18. Buckling modes for the 160 mm high panel (with standard end-diaphragm)
Chapter 4. Preliminary tests on hybrid steel deck

The various diaphragm heights and their connection points can be seen in Figure 4-19.

![Image of diaphragm heights and their connection points]

Figure 4-19. Various diaphragm heights and their connection points

Figure 4-20 show a comparison between the 60 mm and 90 mm diaphragm. The predicted increase in the critical buckling stress is apparent and even more obvious for the 120 mm diaphragm, as seen in Figure 4-21. It can be seen that the values for the lateral-distortional sway mode and the local web buckling mode increase uniformly whereas the values for the top plate buckling mode change insignificantly as the height of the diaphragm is increased. Therefore, it can be concluded that top plate buckling becomes the dominant failure mode, even for very short spans, when the web over the support region is stiffened with a higher and more effective end-diaphragm.

![Graph showing comparison between diaphragm heights]

Figure 4-20. Comparison between 60 and 90 mm high end-diaphragm (TD 160)

![Graph showing comparison between 60 and 120 mm high end-diaphragm]

Figure 4-21. Comparison between 60 and 120 mm high end-diaphragm (TD 160)
4.5 CONCLUSIONS AND OUTCOMES

Over 33 trial specimens with spans up to 8.46 m have been tested as part of product verification for practical applications in commercial projects. The tests allowed the development of an understanding of the flexural behaviour of the hybrid steel deck and a range of possible failure modes (longitudinal shear failure of the TOX connectors in the base panel; top plate buckled under compression; distortion of the web openings; local buckling of the corrugated web over the support; and lateral-distortional sway of the top plate) could be observed. The tests also allowed the development of an appropriate testing apparatus and associated measurement devices. During the course of these preliminary tests, several improvements to the hybrid steel deck have been undertaken to increase its performance. It was also possible to study the influence of base panel thickness, connector spacing, improved longitudinal stiffeners in the top plate, the existence of web openings and various end-diaphragm types regarding failure mode and ultimate moment.

In addition, a numerical investigation (linear-elastic eigenmode extraction) into the effects that an open end-diaphragm would have on the local and global buckling of the cold-formed hybrid cellular deck was undertaken. For the different section heights (TD 110, TD 140 and TD 160) and spans (from 1.5 m to 5.0 m) investigated, a number of failure modes were obtained, viz. lateral-distortional sway, top plate buckling and local web buckling, noting that the model did not account for individual connectors and therefore associated failure modes (longitudinal shear or pull-out of the TOX connectors) could not occur. The results clearly indicated that the existence of the standard end-diaphragm enhances the overall performance of most of the investigated panels. Lateral-distortional sway was the dominant failure mode for short to medium spans whereas longer spans resulted in top plate buckling. Increasing the height of the diaphragm had a positive effect on the maximum critical buckling stress, particularly for the larger section heights, but had only an insignificant effect on the top plate buckling mode.

On the basis of these preliminary tests, decisions were made regarding the significant aspects of the behaviour that would be more closely investigated in the research of this thesis. As a result, the longitudinal shear failure of the TOX connectors in the base panel (BPSH) and the buckling failure of the top plate under compression (TPBU) were identified as the most dominant failure modes and further investigated in an extensive series of flexural tests, as described in the following chapters. The influence that web openings can have on the overall performance of the hybrid steel deck was investigated in a separate study and is not part of this thesis. Selected results of this investigation can be found elsewhere (Patrick et al., 2005). Other failure modes, such as web buckling over the support and lateral-distortional sway of the top plate, appeared to be closely related to the effectiveness of the end-diaphragm and were considered to be of lesser importance since they can readily be eliminated by using appropriate end-diaphragms. Testing, the results of an eigenvalue analysis and subsequent use in practice have shown that the standard open end-diaphragm is suitable for most applications. Therefore, a closer investigation of the influence of end-diaphragms was not considered to be critical at this stage.
In the following extensive series of small-scale tests, the behaviour of TOX connectors used in the hybrid steel deck and loaded in combined shear and tension were studied. The results provided a better understanding of these unusual mechanical connectors, especially when additional tension forces act on the mainly shear-loaded connectors. In the base panel shear tests, the influence of different TOX spacings and material thicknesses were studied. In particular, the possible failure mode where the base panel starts to yield and fractures in tension was further examined. Similarly, the influence of different connector spacings, material thicknesses and material grades were studied in the top plate buckling tests, which then allowed assessing if the top plate was able to yield under compression, or buckled before the yield stress had been reached. The outcome of these investigations may then be used to determine which of the two failure modes would possibly govern in an actual application of the hybrid steel deck.
5.1 GENERAL

In an application like the hybrid steel deck, the main purpose of the mechanical connectors is to resist longitudinal shear. However, having a fastener system that connects the individual elements of the system to a cellular section, separation forces between the elements exist and result in additional tension forces in the connectors. These tension forces might be caused by out-of-plane imperfections of the base panel or the top plate, varying loading arrangements and therefore different distributions of the load or, more importantly, the self-weight of wet concrete applied to the system which is even more critical in the case of having the cellular section filled with concrete and/or using infill panels. In this configuration, the pan of the base panel (or infill panel) takes most of the weight of the wet concrete and wants to separate from the webs, adding tension forces to the mainly shear loaded connectors. Therefore, knowledge of the connectors’ combined shear-tension capacity is fundamental for the design of the hybrid steel deck. The behaviour in combined shear and tension is well known for other types of fasteners, e.g. bolts or screws, and codified in most design standards, including the Australian Steel Standards (AS 4100, 1998; AS/NZS 4600, 2005). However, the behaviour of round press-joints has not been established yet and structural testing is necessary.

5.2 TEST SET-UP

5.2.1 General

Testing individual connectors was the preferred solution since it would give data for a single connector and limit the number of measurement devices. However, there is a need to investigate the strength of the mechanical connectors in the actual application, mainly because isolating a connector could change the boundary conditions and therefore may not represent the behaviour of connectors in an actual panel. For this reason, tests were carried out on cellular sections with one pair of mechanical connectors, i.e. two TOX. The primary requirement in the design of a test rig was to accommodate all
section heights, viz. from 90 mm to 260 mm, irrespective of the material thicknesses used and to investigate the strength of the mechanical connectors under different combinations of direct tensile and shear forces. Both the base panel/web connection and the top plate/web connection can be tested with the same rig but only base panel/web connections were tested in this series.

5.2.2 Test apparatus and specimen preparation

A sophisticated, fully-adjustable test rig was specially developed to test the connector strength of any section height at different loading angles. The test rig consists of two main parts; an upper part (painted green) that connects the webs of the specimen to a load cell attached to the rigid frame of the testing machine; and a lower part (painted blue) connecting the base panel to the moving cross-head of the testing machine. The test-apparatus at various loading angles can be seen in Figure 5-1 and an illustration describing all major elements is given in Figure 5-2.

The lower part consists of a frame of rectangular hollow sections (RHS) joined to a loading fork by M16 bolts allowing the frame to rotate by any angle. The loading angle \( \alpha \) was defined as the angle between the horizontal and the RHS-frame, viz. the base panel. According to this definition, 0-degree means pure tension and 90-degree means pure shear acting on the connection. The angle was recorded with a magnetic protractor attached to the RHS-frame. Eight RHS 50x25x3 were welded together to form the frame and had holes to attach various clamping devices. Three different clamping devices (painted red) were used for fixing the base panel of the test specimens to the frame, viz. a stiffener-clamp, a rib-clamp and a stopper. The flat stiffener-clamp runs through the inside of the cellular section in the vicinity of the inclined webs and clamps the longitudinal stiffener of the base panel to the frame. Additional rib-clamps on the perimeter of the connector prevent hogging bending of the ribs when tension forces are applied. Thick metal angles were used to evenly distribute the local clamping action along the ribs. Although considerable resistance to movement in the longitudinal direction due to friction between the clamps and the base panels was expected, a stopper was integrated in the design of the test rig with the purpose of eliminating any possible movement. The stopper was simply an angle bar rigidly attached to the two stiffener-clamps, preventing the base panel to slip longitudinally. Since the specimens were only 255 mm long, a thick metal plate was used to fill the gap between the location of the angle bar and the edge of the base panel. The exact location of the clamping devices can be seen in Figure 5-3(c), noting that the picture was taken after the connectors failed and the upper part of the test rig including instrumentation was removed.

Two connector plates that are interconnected with two metal rods \( (\varnothing = 27 \text{ mm}) \) and joined to the upper loading fork by M16 bolts form the lower part together with web plates attached to the outside of the inclined webs of the specimen. Large holes at each end of the web plates allow the insertion of the metal rods so that the applied load can be transferred from the upper loading fork to the specimen.
Figure 5-1. Test apparatus at different loading angles

Figure 5-2. Testing Rig (top: front view, bottom: side view)
Chapter 5. Combined shear-tension TOX tests

Conventional TEK screws can be put through a pattern of pre-drilled holes on the web plates to connect the plate with the web. Screws were chosen over other fixing methods mainly because they were easy to remove after each test and required no specially qualified personnel, as would be in the case of welding. The screws are located as low as possible and spread evenly on the web to ensure uniform introduction of the applied load. Both loading forks, the upper and lower, are pin-connected to the rigid frame of the testing machine.

The location of the axis of rotation is crucial for the successful testing of the connection at various loading angles. To ensure the line of action actually goes through the connection, the axis has to be on the same plane in the horizontal as well as the transverse direction as the tested connection. In this way, unwanted bending moments that would reduce the actual failure load can be prevented from being introduced into the connection. This was achieved by locating the centre of the M16 bolt of the lower part exactly on the same plane as the centre of the TOX connector of the test specimen. The location of the M16 bolt of the upper part is highly dependent on how the web plates are attached to the specimen since the distance between the bolt and the metal rods is pre-defined on the connector plates. For this reason, a special assembly rig was used to guarantee the exact position of the web plates relative to the specimen and to simplify the assembling process. A picture of the assembly rig is given in Figure 5-3(b).

Individual test specimens were obtained from continuous panels and had a length of 255 mm with three pairs of connectors. All specimens with the same material combination were retrieved from the same panel to ensure a consistent quality of the TOX connector. The panels were cut into separate segments by using a band saw, as displayed in Figure 5-3(a). Cutting the cellular section caused noticeable vibrations, in particular when the inclined webs were cut. To protect the connectors of interest, in this case the middle pair, from premature loading, adjustable clamps were used on either side of the sawing blade. Once the specimens were separated, the connectors at each end were completely removed with a 12 mm drill bit, leaving only the middle pair intact to be tested. The connectors of the top plate were generally not removed, giving more stability to the section.

![Figure 5-3. Specimen preparation and clamping devices of base panel](a) (b) (c)

5.2.3 Instrumentation and processing of test results

Tests were carried out in an INSTRON universal testing machine (Model No. 6027), equipped with the standard 200 kN load cell, calibrated according to the NATA (National Association of
Testing Authorities, Australia) specifications. For every specimen, load, cross-head movement, slip and separation of each TOX were recorded every second through two synchronized data acquisition systems and all tests, irrespective of loading angle, were carried out with a constant cross-head speed of 0.3 mm/min. The software package MERLIN, installed on a personal computer, was used to record load and cross-head movement of the INSTRON testing machine whereas measurements for slip and separation were captured using a second data acquisition system (Type DATA TAKER DT605, Series 3) and the software package DELOGGER PRO.

Slip between the web and the base panel was measured on both sides, viz. north and south, using 15 mm linear potentiometers (LP’s) glued onto the ribs of the base panel and metal targets glued onto the lower flange of the webs, as seen in Figure 5-4. Specially designed fixtures were made to position the LP’s as low as possible between the rib and the web, thus minimising longitudinal measuring errors due to rotation of the targets caused by uplift and possible bending of the base panel. Since the LP’s were mounted on the specimen, possible slip between the specimen and the testing rig had no effect on the measurements. Separation of the connector on the north and south side was obtained by measuring the lateral displacement of the TOX on the die side (web) as well as on the punch side (base panel) and subtracting the readings from each other since a direct measurement of the separation was not possible. The 15 mm LP’s were therefore directly attached to the RHS-frame to provide an absolute reference point (see Figure 5-4). Measuring displacement on both sides of the TOX ensured that uplift and possible bending of the base panel was eliminated when determining separation of the connector. Uplift values in the range of 0.8 mm and 1.0 mm were measured for base panel thicknesses of 1.00 mm and 0.75 mm respectively. Small metal sheets were glued over the TOX connectors to provide an even area for the LP’s and allow them to slide effortlessly without getting caught at the connector.

![Image](image_url)

**Figure 5-4. LP’s for slip and separation measurements**

5.3 TEST PROGRAM

5.3.1 General

An extensive test program was established covering four different material combinations at five different loading angles. The loading angles ranged from 0-degree (pure tension) up to 90-degree
(pure shear) and two specimens were tested for each loading angle of a specific material combination. Only base panels with a nominal thickness of 0.75 mm and 1.00 mm were tested in this series, noting that base panels with a nominal thickness of 0.55 mm encountered difficulties in manufacturing and are currently not used in actual applications. Each specimen was given a distinct numeric code, e.g. 1007060-2, determined from basic information. The initial three numbers characterize the nominal thickness (in millimetres times 100) of the base panel, e.g. 1.00 mm, followed by two numbers which define the nominal thickness (in millimetres times 100) of the web, e.g. 0.70 mm. The subsequent two numbers represent the loading angle, e.g. 60 degree, and the final number, separated by a dash, states the specimen number, e.g. 2.

5.3.2 Specimen details

The panels tested in this series were custom-made at PREMIER STEEL TECHNOLOGIES Pty. Ltd., NSW, Australia, using the TOX machine described in Chapter 2.4.2. The following tools were used to form the 8 mm TOX:

**Table 5-1. TOX manufacturing tool sets**

<table>
<thead>
<tr>
<th>Material punch side</th>
<th>Material die side</th>
<th>Punch No.</th>
<th>Die No.</th>
<th>Pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.00 mm (G550)</td>
<td>0.70 mm or 0.90 mm (G350)</td>
<td>A60100</td>
<td>BB8012</td>
<td>200 bar</td>
</tr>
<tr>
<td>0.75 mm (G550)</td>
<td>0.70 mm or 0.90 mm (G350)</td>
<td>A60100</td>
<td>BB8012</td>
<td>200 bar</td>
</tr>
<tr>
<td>1.9 mm (HA70T)</td>
<td>0.70 mm or 0.90 mm (G350)</td>
<td>A52100</td>
<td>BC8020</td>
<td>270 bar</td>
</tr>
</tbody>
</table>

Although the test rig is designed to accommodate any section height, it was decided to only use TD 140 sections with a nominal height of 140 mm, webs without corrugations and a 1.9 mm thick top plate. Shape and nominal dimensions of the TD 140 sections are shown in Figure 5-5.

![Figure 5-5. Geometry of TD140 sections](image)

Using a deep section with higher webs made it easier to prepare the specimen but did not have an influence on the performance of the connectors. The thickness of the top plate was chosen arbitrarily and solely on availability since its thickness has no influence on the connectors in the base panel. However, a reasonable thickness was required to provide stability to the cellular section. Flat webs, i.e. without corrugations, were preferred since they allowed a tight contact between the surface of the
Chapter 5. Combined shear-tension TOX tests

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web and the web plate of the test rig, noting that the standard corrugated webs would also be suitable.
The flat webs were obtained by omitting the corrugated moulds in the roll-forming process, leaving
them in their original form.
Key geometries of the lower flange of the web and the TOX connector are illustrated in Figure 5-5
and listed in Table 5-2 for each specimen. For the TOX connector, they include the inside diameter i
on the punch side, the outside diameter o on the die side and the remaining bottom thickness x of the
two connected sheets, called the X-dimension. Key geometries for the lower web flange are the
distance from the centre of the TOX connector to the corner of the web a and to the edge of the
flange b, respectively.
Table 5-2. Key geometries
Code

07570
00-1
00-2
60-1
60-2
69-1
69-2
82-1
82-2
90-1
90-2
07590
00-1
00-2
60-1
60-2
69-1
69-2
82-1
82-2
90-1
90-2
10070
00-1
00-2
60-1
60-2
69-1
69-2
82-1
82-2
90-1
90-2
10090
00-1
00-2
60-1
60-2
68-1
68-2
82-1
82-2
90-1
90-2

South
x
(mm)

i
(mm)

o
(mm)

a
(mm)

b
(mm)

North
x
(mm)

i
(mm)

o
(mm)

a
(mm)

b
(mm)

0.32
0.31
0.29
0.31
0.29
0.30
0.29
0.31
0.33
0.33

6.41
6.42
6.40
6.39
6.40
6.41
6.38
6.37
6.38
6.39

8.18
8.21
8.17
8.15
8.16
8.18
8.14
8.17
8.16
8.17

9.0
11.1
11.0
10.6
10.3
10.3
11.3
11.5
11.0
12.8

16.2
14.2
14.0
15.0
13.7
14.6
13.8
14.0
14.0
12.9

0.28
0.28
0.28
0.29
0.30
0.31
0.30
0.31
0.36
0.36

6.45
6.40
6.38
6.37
6.40
6.42
6.37
6.40
6.36
6.38

8.21
8.18
8.18
8.19
8.17
8.18
8.17
8.16
8.19
8.18

11.4
11.6
9.4
11.7
9.8
11.1
8.8
10.2
9.9
9.5

13.6
13.1
16.0
13.0
15.1
14.0
16.7
14.8
15.0
15.2

0.39
0.40
0.42
0.42
0.40
0.42
0.44
0.43
0.42
0.42

6.45
6.45
6.45
6.43
6.45
6.45
6.43
6.42
6.44
6.45

8.16
8.19
8.18
8.18
8.20
8.15
8.17
8.22
8.18
8.15

10.2
10.5
10.3
11.7
11.4
10.2
9.9
10.3
9.6
10.5

13.9
13.2
13.3
12.3
12.5
13.7
13.2
15.8
14.2
13.3

0.40
0.40
0.43
0.42
0.42
0.42
0.42
0.42
0.44
0.48

6.43
6.48
6.45
6.45
6.47
6.45
6.47
6.48
6.45
6.47

8.17
8.18
8.20
8.18
8.18
8.18
8.17
8.20
8.15
8.20

9.7
9.8
10.4
10.7
9.5
10.9
9.7
9.0
9.7
10.7

13.8
14.0
13.6
13.4
13.9
13.2
13.7
15.1
13.8
13.1

0.53
0.52
0.52
0.55
0.52
0.52
0.52
0.51
0.54
0.53

6.38
6.38
6.40
6.36
6.41
6.40
6.41
6.38
6.41
6.38

8.18
8.18
8.16
8.18
8.26
8.22
8.18
8.17
8.17
8.16

11.3
12.0
11.2
12.0
11.4
12.1
12.3
12.1
12.2
11.2

13.5
13.4
13.2
12.7
14.2
13.0
13.0
12.7
12.6
13.5

0.57
0.52
0.56
0.54
0.53
0.53
0.51
0.52
0.54
0.54

6.39
6.38
6.38
6.40
6.42
6.40
6.40
6.38
6.42
6.41

8.17
8.18
8.19
8.18
8.22
8.26
8.18
8.19
8.17
8.17

11.0
10.4
9.8
10.6
11.0
12.5
12.4
10.3
10.2
9.2

11.3
14.4
14.9
14.1
13.8
12.6
13.1
15.1
14.0
15.7

0.65
0.62
0.60
0.60
0.64
0.59
0.62
0.60
0.60
0.62

6.45
6.43
6.42
6.43
6.43
6.45
6.44
6.45
6.48
6.41

8.18
8.17
8.20
8.19
8.18
8.17
8.18
8.20
8.18
8.18

8.8
11.0
12.8
10.0
11.1
13.0
11.0
12.5
12.4
10.6

13.6
12.8
11.1
13.3
13.0
10.4
12.9
11.3
11.4
13.0

0.66
0.63
0.61
0.62
0.61
0.61
0.64
0.62
0.60
0.62

6.45
6.45
6.43
6.42
6.45
6.45
6.45
6.45
6.45
6.45

8.19
8.20
8.20
8.18
8.20
8.18
8.18
8.20
8.19
8.16

10.5
10.6
10.8
11.3
10.8
11.1
9.9
11.0
10.7
10.1

13.6
13.4
13.0
12.3
12.9
12.6
14.1
12.8
12.5
13.4


Chapter 5. Combined shear-tension TOX tests

All test specimens were approximately 255 mm long and carefully cut from continuous panels, as described before. The panels had a constant connector spacing of 86 mm and the peculiar spacing was essential to obtain a non-staggered TOX arrangement for the base panel where the connectors on the north and south side face each other. Having a non-staggered TOX arrangement was very important for testing at different loading angles since it made sure that the axis of rotation of the test rig goes through the centre of both connectors. Otherwise, the offset between the connectors would have introduced additional moments and reduce the actual failure load due to prying action. It should be noted that for testing in pure shear (90 degree), a staggered TOX arrangement can also be used without reducing the actual failure load. With a length of 255 mm, each specimen originally contained three pairs of connectors and the connectors at each end had to be removed with a 12 mm drill bit so that only the middle pair was left intact to be tested.

5.3.3 Mechanical properties

Tensile testing of coupons was carried out to determine the mechanical properties of the material used in the test series, following the guidelines of the Australian Standard (AS 1391, 1991). Material for the base panel and the web were directly obtained from the same coil used to manufacture the specimen in this series. Material for the top plate was considered to be non-influential in regard to the performance of the connectors between the base panel and web. Therefore, the mechanical properties for the top plate material were not determined. All materials were tested in the longitudinal, i.e. rolling, direction. Testing of material transverse to the rolling direction was considered to be not of importance since loads in the combined shear-tension tests were only applied in the longitudinal direction. Properties of the coated material were based on the base metal thickness (b.m.t.) although the coupons were tested in their original coated state. A detailed description of the test method and all test results can be found in Appendix A. The average results for the different materials used in this test series are listed in Table 5-3.

### Table 5-3. Average tensile coupon test results

<table>
<thead>
<tr>
<th>Coupon</th>
<th>Coating</th>
<th>$t_c$ (mm)</th>
<th>$t_b$ (mm)</th>
<th>$f_{t^*}$ (MPa)</th>
<th>$\varepsilon_{0.2}^*$ (%)</th>
<th>$f_{u^*}$ (MPa)</th>
<th>$E^*$ (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Base panel material (G550)</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>G550/1.00-1</td>
<td>coated</td>
<td>1.06</td>
<td>0.99</td>
<td>647</td>
<td>0.53</td>
<td>660</td>
<td>234.8</td>
</tr>
<tr>
<td>G550/0.75-1</td>
<td>coated</td>
<td>0.82</td>
<td>0.74</td>
<td>710</td>
<td>0.55</td>
<td>713</td>
<td>240.7</td>
</tr>
<tr>
<td><strong>Web material (G350)</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>G350/0.9</td>
<td>coated</td>
<td>0.93</td>
<td>0.88</td>
<td>388</td>
<td>0.43</td>
<td>486</td>
<td>226.5</td>
</tr>
<tr>
<td>G350/0.7</td>
<td>coated</td>
<td>0.76</td>
<td>0.70</td>
<td>412</td>
<td>0.44</td>
<td>510</td>
<td>222.4</td>
</tr>
</tbody>
</table>

* values based on b.m.t.

5.4 TEST RESULTS

5.4.1 General

A total of 40 specimens have been tested in this series. Since two data acquisition systems were used during testing, both data streams had to be synchronized. Pin connections between the rigid loading frame and the test apparatus allowed slack therefore load was determined from the applied load, taking account of the self-weight of the testing apparatus. All results are stated as load for two connectors, viz. on the north and south side. However, to obtain a reasonable estimate of the force on
each connector, the load can be simply divided by two since a pair of connectors was always tested at the same time and special care was taken to load the specimens equally.

5.4.2 Failure modes

Three different failure modes were encountered during testing, namely shear (mode I), pullout (mode II) and combined shear-pullout (mode III). Figure 5-6 shows an example of each failure, noting that the web has been shifted to the side for better visibility.

![Figure 5-6. Failure modes]

The failure modes were highly dependent on the applied loading angle. All specimens tested in pure shear (90 degree) failed in mode I, noting that the test was usually stopped after the first connector failed and subsequently created an unequal loading arrangement. In contrast to the failure modes described in Chapter 3.1.5, no pull-over failure was observed in this series. This can be simply explained by the fact that, with this test rig and its clamping devices, no rotation or bending of the base panel was possible compared to the back-to-back test set-up used in Chapter 3.1.2. There, slight rotation of the joints could still occur with the relatively flexible strips of thin sheet metal back-to-back that allowed the joints to fail in a pull-over rather than in shear. Rotation of the press-joints in the base panel of an actual cellular section would require a lot of local deformation around the joint and, under the given loads, a shear failure of the neck of the press-joint is more likely, in particular when the base panel is in tension due to flexural bending. The pure shear behaviour of the press-joints was therefore better simulated with the modified test set-up or the current test rig.

All specimens tested in pure tension (0 degree) experienced a pullout failure (mode II). In this mode of failure, the sheet on the die side of the joint deforms and the joint subsequently opens as a press stud, losing the interlocking of sheets. The clinch on the punch side stays intact without fracturing the neck of the joint. Naturally, this mode is the weakest with the lowest failure load since it requires deformation of the ductile G350 material on the die side rather than fracture of the high-strength G550 material on the punch side of the joint. The failure load was also influenced by the distinct shape of the cellular section. Since the load was applied through the webs, the lower flange of the web acted like a cantilever, putting additional stress onto the connector due to the prying effects of the outstand of the flange. The pull-out failure mechanism is documented in Figure 5-7 at three different stages where the prying effect is clearly visible.
Chapter 5. Combined shear-tension TOX tests

On a few occasions, a combination of a shear and a pullout failure (mode III) occurred. This mode is characterized by shearing of one side of the joint first after which the joint opens and the remaining parts pull out. It was only observed for material combination 07590 at a loading angle of 69 degree and once for material combination 10070 at 82 degree. At these loading angles, the failure mode usually tends to switch from mode I to mode II. Initial imperfections, such as miniature cracks in the neck of the joint, possibly contributed to the observed combined shear-pullout failures.

5.4.3 Results

All test results for the 0.75 mm and 1.00 mm base panel are summarized in Table 5-4. Ultimate load \( P_u \) was defined as the maximum load measured during testing. Values are given for slip \( s_u \) and separation \( A_u \) when the ultimate load was reached. Since slip and separation was measured on the north and south side, only the higher value of both sides is listed and marked with a star (*) when measured on the north side. The list also contains both the fracture load \( P_f \) when one of the connectors failed and the observed fracture mode. In the case where one connector failed first, the test was usually stopped leaving the other connector intact and no failure mode was therefore recorded. This is then indicated with a dash.

The influence of the loading angle on the ultimate load is noticeable with the highest load reached at an angle of 90 degree, i.e. pure shear, and continuously decreasing with smaller angles. At 0 degree (pure tension), the load dropped by 77 % for material combinations 07570 and 10090, by 67 % for 07590 and by 85 % for 10070 compared to the load at 90 degree. The results are similar to those of cross-tension tests carried out on specimens with the same material of equal thickness on the punch and die side, demonstrating a loss of 50 % to 70 % in pullout strength compared to their capacity in pure shear (Liebig et al., 1989). However, individual results are highly dependent on the internal geometry of the press-joint under investigation, in particular on the extent of interlocking between the two sheets achieved with the TOX tool set used in manufacture. In the case of the hybrid steel deck, the results are further reduced due to the prying effect caused by the outstand of the lower web flanges. Changes in the loading angle did not have much influence in the range from 0 degree to 60 degree whereas even small changes in loading direction between 60 degree and 90 degree altered the ultimate load considerably.
Table 5-4. Test results

<table>
<thead>
<tr>
<th>Code</th>
<th>$\alpha$</th>
<th>$P_s$ (kN)</th>
<th>$s_u$ (mm)</th>
<th>$\Delta_u$ (mm)</th>
<th>$P_f$ (kN)</th>
<th>$P_f / P_u$ (-)</th>
<th>Failure mode north / south</th>
</tr>
</thead>
<tbody>
<tr>
<td>07570</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>00-1</td>
<td>0</td>
<td>1.58</td>
<td>0.26*</td>
<td>1.50</td>
<td>1.44</td>
<td>0.91</td>
<td>- / II</td>
</tr>
<tr>
<td>00-2</td>
<td>0</td>
<td>1.49</td>
<td>0.12</td>
<td>1.50</td>
<td>1.27</td>
<td>0.85</td>
<td>- / II</td>
</tr>
<tr>
<td>60-1</td>
<td>60</td>
<td>2.72</td>
<td>0.07</td>
<td>1.29</td>
<td>2.39</td>
<td>0.88</td>
<td>- / II</td>
</tr>
<tr>
<td>60-2</td>
<td>60</td>
<td>2.71</td>
<td>0.17*</td>
<td>1.40</td>
<td>2.50</td>
<td>0.92</td>
<td>- / II</td>
</tr>
<tr>
<td>69-1</td>
<td>69</td>
<td>3.67</td>
<td>0.44</td>
<td>1.74</td>
<td>3.55</td>
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<td>- / II</td>
</tr>
<tr>
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<td>3.87</td>
<td>0.43*</td>
<td>1.26*</td>
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<td>0.96</td>
<td>II / -</td>
</tr>
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<td>1.22*</td>
<td>5.78</td>
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<td>I / -</td>
</tr>
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</tr>
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<td>6.76</td>
<td>1.00</td>
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</tr>
<tr>
<td>90-2</td>
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<td>0.59*</td>
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<td>I / -</td>
</tr>
<tr>
<td>07590</td>
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<td></td>
<td></td>
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</tr>
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</tr>
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<td>0.06</td>
<td>1.97</td>
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<td>0.93</td>
<td>- / II</td>
</tr>
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</tr>
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<td>2.12*</td>
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<td>1.74*</td>
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<td>5.68</td>
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<td>0.70</td>
<td>5.67</td>
<td>0.99</td>
<td>I / I</td>
</tr>
<tr>
<td>90-1</td>
<td>90</td>
<td>5.80</td>
<td>0.46*</td>
<td>0.18*</td>
<td>5.77</td>
<td>1.00</td>
<td>I / -</td>
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<td>90-2</td>
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<td>6.41</td>
<td>0.36*</td>
<td>0.15*</td>
<td>6.40</td>
<td>0.99</td>
<td>I / I</td>
</tr>
<tr>
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</tr>
<tr>
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<td>0.10*</td>
<td>1.41</td>
<td>1.00</td>
<td>0.87</td>
<td>II / -</td>
</tr>
<tr>
<td>00-2</td>
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<td>0.20*</td>
<td>1.38</td>
<td>1.09</td>
<td>0.89</td>
<td>II / -</td>
</tr>
<tr>
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<td>0.00</td>
<td>1.09</td>
<td>1.88</td>
<td>0.90</td>
<td>- / II</td>
</tr>
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<td>- / II</td>
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</tr>
<tr>
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<td>0.33*</td>
<td>2.61</td>
<td>6.22</td>
<td>0.96</td>
<td>II / II</td>
</tr>
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<td>0.92</td>
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</tr>
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<td>90-1</td>
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<td>1.27*</td>
<td>0.17*</td>
<td>8.00</td>
<td>0.97</td>
<td>I / II</td>
</tr>
<tr>
<td>90-2</td>
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<td>1.39*</td>
<td>0.48</td>
<td>8.17</td>
<td>0.98</td>
<td>I / II</td>
</tr>
<tr>
<td>10090</td>
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</tr>
<tr>
<td>00-1</td>
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<td>0.80</td>
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<td>0.85</td>
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</tr>
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<td>0.14</td>
<td>1.08</td>
<td>3.09</td>
<td>0.93</td>
<td>- / II</td>
</tr>
<tr>
<td>60-2</td>
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<td>3.22</td>
<td>0.08*</td>
<td>1.08</td>
<td>2.99</td>
<td>0.93</td>
<td>II / -</td>
</tr>
<tr>
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<td>0.60*</td>
<td>1.80</td>
<td>4.56</td>
<td>0.95</td>
<td>II / -</td>
</tr>
<tr>
<td>68-2</td>
<td>68</td>
<td>4.50</td>
<td>0.57*</td>
<td>1.49*</td>
<td>4.32</td>
<td>0.96</td>
<td>II / -</td>
</tr>
<tr>
<td>82-1</td>
<td>82</td>
<td>7.25</td>
<td>0.42*</td>
<td>0.69*</td>
<td>7.24</td>
<td>1.00</td>
<td>I / -</td>
</tr>
<tr>
<td>82-2</td>
<td>82</td>
<td>6.98</td>
<td>0.56*</td>
<td>0.80</td>
<td>6.95</td>
<td>1.00</td>
<td>I / -</td>
</tr>
<tr>
<td>90-1</td>
<td>90</td>
<td>7.67</td>
<td>0.41*</td>
<td>0.23*</td>
<td>7.66</td>
<td>1.00</td>
<td>I / -</td>
</tr>
<tr>
<td>90-2</td>
<td>90</td>
<td>7.71</td>
<td>0.54*</td>
<td>0.23*</td>
<td>7.65</td>
<td>0.99</td>
<td>I / -</td>
</tr>
</tbody>
</table>

* measured on north side

As already observed and explained in the shear tests of Chapter 3.1.5, joints with a 1.00 mm thick base panel reached higher loads in pure shear than joints with a 0.75 mm base panel, and material combinations with a 0.70 mm thick web are slightly stronger than combinations with 0.90 mm webs. Tests carried out under pure tension revealed different relationships for ultimate load between the material thickness of the punch side and the die side. Generally, material combinations with the thicker 0.90 mm web were stronger than the 0.70 mm webs, irrespective of the thickness of the base panel. However, for a given web thickness, material combinations with the thinner 0.75 mm base panel were stronger than the 1.00 mm base panels. Although these relationships seem to be odd in the first place, a closer look at the failure mechanism can give an explanation. The pullout failure was characterized by an opening of the joint due to the deformations on the die side. It appears that thicker
material on the die side, i.e. web side, is more difficult to deform and therefore higher pullout loads were achieved. Considering the same TOX tool set and pressure was used for all material combinations, the material of the thinner base panel, i.e. 0.75 mm was able to flow further in radial direction in order to fill the refluxing hole of the die, thus forming a larger undercut between the sheets. The size of the undercut is an important variable for the pullout capacity of a press-joint and is mainly responsible for the interlocking between the sheets.

Figure 5-8 displays ultimate load versus applied loading angle of all tested specimens. The relationship can be approximated with a bilinear model for each material combination, with a gradual increase in load from 0 degree to 60 degree and a steeper increase in load from 60 degree to 90 degree. This shows that even minor changes in loading direction from loading at 90 degree can reduce the ultimate load significantly. The ultimate load at 60 degree is around 40 % of the load at 90 degrees for material combination 07570, 55 % for combination 07590, 25 % for combination 10070 and 43 % for combination 10090. The slopes for the material combinations with 0.70 mm webs are generally steeper due to their higher capacity in pure shear and lower capacity in pure tension.

Typical load-slip and load-separation curves at three loading angles (0 degree, 82 degree and 90 degree) are given in Figure 5-9 for material combination 07570 and in Figure 5-10 for combination 10070. The individual tests used in this graph are indicated in bold type for the code in Table 5-4. Results for all tests carried out in this series can be found in Appendix D. The greater the angle, the higher the proportion of shear that gets transferred into the connection, resulting generally in bigger slip values and less separation of the TOX. Usually, the shape of the curves for the north side differs slightly from the south side after they reach a certain load level. At this point one connector starts to deform more than the other and the load gets distributed unevenly until the connector reaches its limit and fails.
In this complex test rig, it was not possible, due to space considerations, to measure slips directly at or adjacent to the TOX connectors. Instead the slip was measured some distance away from the connector. Moreover, the slip LP’s were mounted on the ribs of the base panel above the level of the TOX with the measuring target mounted on the lower web flange at the level of the TOX. Therefore any rotations in the test panel could affect the LP measurements which may explain the initial negative slips measured for some specimens. This effect becomes less significant when slips are large. However, the slip results do indicate the relative behaviour of TOX connectors tested at different shear-tension configurations where tension reduces the overall slip at failure.
Similar to the shear tests of Chapter 3.1.5, very high connector stiffnesses were observed in pure shear at the beginning of loading. The initial connector stiffness is only valid up to a certain load level (around 60% of the ultimate load) when it starts to flatten out and asymptotes to the horizontal. At failure, the connection suddenly loses its load carrying capacity, clearly noticeable in the abrupt drop of the load curve. Once a connection reaches its ultimate load, slip and separation values continue to grow until the connector finally fails at a slightly reduced load. The difference between ultimate load and failure load is less than 3% in pure shear, and somewhere between 10 to 15% in pure tension, with the exception of material combination 07590 with 26%.

Figure 5-10. Material combination 10070
5.5 INTERACTION

To establish an interactive relationship for the round press-joints in combined shear-tension, the load was divided into its shear and tensile components since the test rig was only able to measure the applied load in one direction. This was achieved by using trigonometric functions and the applied loading angle. The values of each test were then non-dimensionalised using the maximum mean values from two specimens for pure shear \( V_{\text{max}} \) and pure tension \( N_{\text{max}} \) for each material combination which are listed in Table 5-5.

**Table 5-5. Maximum values of each material combination (mean for 2 TOX)**

<table>
<thead>
<tr>
<th>Code</th>
<th>( V_{\text{max}} ) (kN)</th>
<th>( N_{\text{max}} ) (kN)</th>
<th>Code</th>
<th>( V_{\text{max}} ) (kN)</th>
<th>( N_{\text{max}} ) (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>07570</td>
<td>6.59</td>
<td>1.54</td>
<td>10070</td>
<td>8.27</td>
<td>1.19</td>
</tr>
<tr>
<td>07590</td>
<td>6.13</td>
<td>2.00</td>
<td>10090</td>
<td>7.69</td>
<td>1.74</td>
</tr>
</tbody>
</table>

Once the normalized results of all four material combinations are applied to one graph, a distinct relationship between shear and tension values is noticeable (see Figure 5-11). Various models exist to describe the interaction of combined shear-tension for other types of fasteners. Probably the most common approach is expressed in equation (5-1).

\[
\left( \frac{N}{N_{\text{max}}} \right)^n + \left( \frac{V}{V_{\text{max}}} \right)^n = 1
\]

(5-1)

Depending on the exponent \( n \), different relationships can be established. An exponent equal to one reveals a linear relationship and is used in the European Steel Design Standard (EC 3-1.3, 1996) for a fastener loaded in combined shear and tension. The Australian Steel Design Standards (AS 4100, 1998; AS/NZS 4600, 2005) use an exponent equal to two to design bolts in combined shear and tension which essentially describes a circular relationship. Assuming no interaction between shear and tension exists at all, the exponent would be infinitely large.

A different approach, based on a tri-linear model, was taken in the British Steel Design Standard (BS 5950:Part 1, 1985) for the interaction of bolts in combined shear and tension as expressed in equation (5-2).

\[
\frac{N}{N_{\text{max}}} + \frac{V}{V_{\text{max}}} \leq 1.4 \quad \text{with the limits:} \quad N \leq N_{\text{max}} \quad V \leq V_{\text{max}}
\]

(5-2)

It can be seen that the test results fit more closely to the BS 5950 approach or the Australian/New Zealand approach with \( n = 2 \). However, if one of these curves was used as the basis for design, the variation of the results from this curve would need to be taken into account statistically (Pham and Hogan, 1985) to determine appropriate capacity reduction factors \( \phi \) (as used in the Australian or North American Code) or partial material factors \( \gamma \) (as used in the European Code).
Figure 5-11. Interaction between shear and tension for round press-joints

5.6 CONCLUSION

A total of 40 specimens have been tested at various loading angles. Four different material combinations were under investigation resulting in the highest failure loads for pure shear (90 degree). Changes in the loading angle were most significant between 60 degree and 90 degree, and less significant in the range from 0 degree to 60 degree. Combinations with thicker punch side material achieved higher ultimate loads in pure shear which was not necessarily the case in pure tension (0 degree). Depending on the material combination, load reductions of 67% to 85% were registered in pure tension compared to the values in pure shear. Individual results were highly dependent on the internal geometry of the press-joint under investigation, noting that the same TOX tool set was used in the manufacture of the specimens for all combinations. Tests involving tension were also influenced by the distinct shape of the cellular section, putting additional stress onto the connector due to prying effects of the lower web flanges. Interactive relationships between shear and tension have been proposed. Before these could be adopted for design, a statistical analysis would need to be carried out to determine appropriate capacity reduction factors $\phi$ or partial material factors $\gamma$ depending on the design approach.
6.1 GENERAL

During preliminary testing of the hybrid steel deck, various major failure modes possible during flexural bending have been identified, with a shear failure of the TOX connectors in the base panel as one of them (Chapter 4). Although clinch connectors have already been used as structural components in the building industry, the hybrid steel deck is, to the author’s knowledge, the first product that uses round press-joints to connect various structural elements subject to bending over a considerably length. The hybrid steel deck could therefore be described as a composite steel section with the TOX press-joints acting as shear connectors, similar to a composite steel-concrete beam. In order to more closely study the longitudinal shear behaviour of these press-joints, which essentially have a very limited slip capacity, a programme of laboratory tests has been carried out. The results were then used to develop models that allow the behaviour of the panel to be predicted with regards to deflection and strength. The models allow for partial shear connection where the flexural strength of the hybrid steel deck is governed by the strength of the TOX connectors rather than the strength of the steel sheeting used in the panel components.

6.2 TEST SET-UP

6.2.1 General

The primary requirement in the design of this test set-up was to provide a sufficient length of constant shear without causing secondary failure modes and hence negatively influencing the longitudinal shear behaviour of the TOX connectors. A single span with a point load at midspan would be the optimal solution, but not reasonable due to the expected high load level, in particular when fracture of the high-strength base panel occurred. Therefore a two point load was chosen to better distribute the applied load, reducing the likelihood of local failures at the loading points.
6.2.2 Test apparatus

All tests of this series have been conducted as single span tests with a distributed two-point load near mid-span and roller bearings as supports to allow horizontal movement of the specimen to occur freely during bending. A 200 mm wide and 10 mm thick plate was fitted over the roller supports to better distribute the reaction forces. The standard loading rig consisting of a crossbeam, two loading fingers and a loading bracket, as described in Chapter 4.1, has been used. The distance between the two loading points was 750 mm, and each loading point was supplied with two standard loading rigs to avoid concentrated loading of the sheeting and consequently premature failure of the press-joints. The loading rigs were rigidly connected to a spreader beam, at a distance of 150 mm, which was attached to the loading beam via a pivot point. This arrangement provided 1,800 mm of constant shear region and 750 mm overhang at each end. A schematic view of the loading arrangement can be seen in Figure 6-1, whereas Figure 6-2 shows the actual test apparatus.

![Figure 6-1. Loading frame with distributed two-point load – Side View](image)

![Figure 6-2. Test apparatus](image)
Small metal packers, approximately 25 mm long and 3 mm thick, were used in between the press-joints to firstly provide an even area for the loading brackets and secondly not to directly load the connectors as this could possibly influence their structural performance. The jack load was applied through a 1000 kN capacity MTS hydraulic actuator with a 250 mm stroke that allowed the use of pre-programmed stroke rates. Tests were carried out under position-control and the rates were usually in the range of 4 to 8 mm/min during the loading process and 6 to 10 mm/min during the unloading process.

### 6.2.3 Instrumentation and processing of test results

With expected loads below 50 kN, an external 100 kN load cell (Type TML-TCLM-10B) was used to measure the total applied jack load. The load cell was previously calibrated according to the NATA (National Association of Testing Authorities, Australia) specifications. Linear potentiometers (LP’s) of the type SAKAE-S18FLPA100R with a range of 100 mm were used to measure vertical deflection near the loading points at three distinctive points across the base panel, and on the north side at mid-span. Longitudinal slip between the base panel and the web was measured on the north side of both shear spans using 15 mm LP’s (SAKAЕ –S18FLPA15R) hot-glued onto the ribs of the base panel and targets hot-glued between the connectors onto the lower flange of the web. Additionally, up to five strain gauges (120 Ω resistance gauges, produced by TOKYO SOKKI) were attached to the bottom of the base panel at mid-span, using M-line accessories and following the manufacturer’s recommended procedures and guidelines. Deflection, slip, load and strain gauge measurements were captured using a data acquisition system (DATA TAKER DT605, Series 3 for strain gauge measurement, and DATA TAKER DT800, for remaining measurements) and the software package DELOGGER PRO. Instrumentation of the test panel is illustrated in Figure 6-3 and details are shown in Figure 6-4.

![Figure 6-3. Instrumentation of test panel](image-url)
6.3 TEST PROGRAM

6.3.1 General

Various secondary failure modes were considered and measures were taken to avoid them. A top plate failure was prevented by using a relatively thick top plate of 2.5 mm combined with stiffening elements, in this case 20 mm reinforcing bars, attached to it. Since the manufacturing process of the test panels only allowed equal numbers of TOX connectors in the top and bottom of the closed section, additional connectors were manually attached to the top plate to prevent a premature pull-out failure of the top plate connectors. The lowest section height with a nominal height of 90 mm and a solid web, i.e. no web penetrations, was chosen to avoid web buckling. The stocky section in combination with end-diaphragms made the lateral-distortional sway mode unlikely, leaving shear failure of the base panel connectors or fracture of the base panel as the desired failure mode.

Three different material combinations were tested at various connector spacings. Only G550 base panels with a nominal thickness of 0.75 mm and 1.00 mm were tested in this series, noting that base panels with a nominal thickness of 0.55 mm encountered difficulties in manufacturing and are currently not used in actual applications. Other base panel steel grades were not considered in this test series, since lower grades reduce the tensile capacity of the sheet in a composite slab and were consequently not used for practical applications in commercial projects. The tests were focused on a nominal web thickness of 0.70 mm which represents the standard web thickness in actual applications. However, the influence of a thicker web, i.e. 0.90 mm, was also investigated but limited to a base panel thickness of 1.00 mm. The connector spacing varied from as close as to achieve complete shear connection in the base panel, i.e. fracture of the base panel, up to approximately 50% partial shear connection. All panels of this test series had to be manufactured well before the actual testing to avoid conflicts with the commercial manufacturing line. Therefore, various assumptions regarding specimen details were made based on the knowledge at that time. For example, the nominated panel length was 6.0 m but preliminary testing revealed exceptional deflections for the 90 mm section height, and the span was therefore reduced to 4.5 m. This provided 1.8 m of constant shear region and 0.75 m overhang at each end of the panel. This allowed testing of two different configurations, A and F. Configuration A were panels with end anchorage, viz. the connectors at the overhang were intact, simulating a cantilever in real applications, and configuration F (free end) with the connectors being removed which is the standard scenario.
A distinct alpha-numeric code, e.g. 10070100-A, determined from basic information was given to each specimen. The initial three numbers characterize the nominal thickness of the base panel (in millimetres times 100), e.g. 1.00 mm, followed by the nominal thickness of the web (in millimetres times 100), e.g. 0.70 mm. The subsequent two to three numbers represent the constant connector spacing of the base panel in millimetres, e.g. 100 mm. The final letter, separated by a dash, states the configuration, e.g. end anchorage (A) or free (F).

### 6.3.2 Specimen details

The panels tested in this series were custom-made at PREMIER STEEL TECHNOLOGIES Pty. Ltd., NSW, Australia, using the TOX machine described in Chapter 2.4.2. The following tools were used to form the 8 mm TOX:

<table>
<thead>
<tr>
<th>Material punch side</th>
<th>Material die side</th>
<th>Punch No.</th>
<th>Die No.</th>
<th>Pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.50 mm (Blackform)</td>
<td>0.70 mm or 0.90 mm (G350)</td>
<td>A52100</td>
<td>BC8020</td>
<td>270 bar</td>
</tr>
<tr>
<td>1.00 mm (G550)</td>
<td>0.70 mm or 0.90 mm (G350)</td>
<td>A60100</td>
<td>BB8012</td>
<td>200 bar</td>
</tr>
<tr>
<td>0.75 mm (G550)</td>
<td>0.70 mm (G350)</td>
<td>A60100</td>
<td>BB8012</td>
<td>200 bar</td>
</tr>
</tbody>
</table>

All sections had a nominal height of 90 mm with a 2.5 mm thick top plate. A shallow section height with a thick top plate material was chosen to reduce the possibility of secondary failure modes such as web buckling or top plate buckling. Shape and nominal dimensions of the sections are shown in Figure 6-5.

![Figure 6-5. Nominal dimensions of TD90 sections](image-url)
During manufacturing, several modifications were made to the panels:

- In contrast to the standard practice, no additional TOX were concentrated at the ends of the panels, to ensure a constant TOX spacing in the shear region. However, the first connector pair at each panel end was spaced closer due to the manufacturing process and intended to be located past the supports.

- Thin-gauge, pressed open diaphragms were positioned over the intended support points (50 mm from the panel ends) as well as at the intended loading points (2,250 mm from each panel end) and only attached to the webs using self-piercing rivet (HENROB). This is again in contrast to the standard manufacturing procedure which also connects the diaphragms to the base panel. However, this would have caused additional longitudinal restraint.

- Additional self-piercing rivets (HENROB) were placed between the regular (but sometimes widely) spaced TOX connectors of the top plate, in order to prevent potential secondary failures in the top plate since equal numbers and spacing of connectors are made in the top and the bottom of the panel due to the manufacturing equipment used.

A total of 18 specimens have been tested and TOX spacing in the base panel was kept constant over the length for each specimen, except for the ends with usually two connectors at a spacing of 20 mm. Table 6-2 summarizes the number of connectors for all tested specimens. The first number is the number of TOX between the loading point and the support, followed by the number of connectors located in the overhang (the number after the dash represents the connectors with a 20 mm spacing). It should be noted that the number of connections are not always equal on the north and south side of the panel, due to a staggered TOX arrangement, which was unavoidable in the manufacturing process.

**Table 6-2. Number of connectors**

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Spacing (mm)</th>
<th>Number of connectors</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>East-North</td>
<td>East-South</td>
</tr>
<tr>
<td>0757060-A</td>
<td>60</td>
<td>29 / 11-3</td>
</tr>
<tr>
<td>0757080-A</td>
<td>80</td>
<td>21 / 8-2</td>
</tr>
<tr>
<td>0757080-A*</td>
<td>80</td>
<td>22 / 8-2</td>
</tr>
<tr>
<td>0757080-F*</td>
<td>80</td>
<td>22 / 1</td>
</tr>
<tr>
<td>07570100-A</td>
<td>100</td>
<td>18 / 7-2</td>
</tr>
<tr>
<td>07570100-F</td>
<td>100</td>
<td>17 / 1</td>
</tr>
<tr>
<td>0757120-A</td>
<td>120</td>
<td>14 / 6-2</td>
</tr>
<tr>
<td>0757120-A*</td>
<td>120</td>
<td>14 / 6-2</td>
</tr>
<tr>
<td>0757120-F*</td>
<td>120</td>
<td>14 / 2</td>
</tr>
<tr>
<td>1007055-A</td>
<td>55</td>
<td>31 / 13-2</td>
</tr>
<tr>
<td>1007075-F*</td>
<td>75</td>
<td>22 / 2</td>
</tr>
<tr>
<td>1007085-A</td>
<td>85</td>
<td>20 / 8-2</td>
</tr>
<tr>
<td>10070100-A</td>
<td>100</td>
<td>18 / 6-2</td>
</tr>
<tr>
<td>10070100-F</td>
<td>100</td>
<td>18 / 1</td>
</tr>
<tr>
<td>1007120-A</td>
<td>120</td>
<td>14 / 6-2</td>
</tr>
<tr>
<td>1007120-F</td>
<td>120</td>
<td>14 / 2</td>
</tr>
<tr>
<td>1009055-A</td>
<td>55</td>
<td>31 / 12-3</td>
</tr>
<tr>
<td>1009075-F</td>
<td>75</td>
<td>23 / 2</td>
</tr>
<tr>
<td>1009100-F</td>
<td>100</td>
<td>18 / 1</td>
</tr>
<tr>
<td>1009120-F</td>
<td>120</td>
<td>14 / 2</td>
</tr>
</tbody>
</table>

Three specimens (0757080*, 07570120*, 1007075*), indicated with a star, were first tested under configuration A up to a certain initial load level then fully unloaded and in-situ transformed into
configuration F to then be tested until failure. The respective initial load levels were 75%, 91% and 81% of their maximum load under configuration F.

The quality of a TOX joint can be established in a non-destructive way by measuring the material thickness in the middle of the base of the joint, called the X-dimension, and comparing it to the manufacturers’ recommendations. The measured values as listed in Table 6-3 tend to be less than the recommended values, but were generally within the 15% tolerance allowed by the manufacturer (TOX Pressotechnik, 2002). Micrographs of the TOX joints used in this test program are shown in Figure 6-6. The micrographs reveal a very complicated internal geometry of the press-joins. It can be seen, that the neck thickness varies depending on the material combination, where a thicker punch side material results in a larger neck thickness. Irrespective of the material combinations used, the size of the undercut, and therefore the interlock between the joining parts, is relatively small.

Figure 6-6. Micrographs of various TOX joints
Chapter 6. Base panel shear tests

Table 6-3. Average bottom thickness of TOX joints

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Recommended (mm)</th>
<th>East-North (mm)</th>
<th>East-South (mm)</th>
<th>West-North (mm)</th>
<th>West-South (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0757060-A</td>
<td>0.40</td>
<td>0.42</td>
<td>0.41</td>
<td>0.43</td>
<td>0.41</td>
</tr>
<tr>
<td>0757080-A</td>
<td>0.40</td>
<td>0.48</td>
<td>0.44</td>
<td>0.47</td>
<td>0.43</td>
</tr>
<tr>
<td>0757080-A/F*</td>
<td>0.40</td>
<td>0.42</td>
<td>0.42</td>
<td>0.41</td>
<td>0.41</td>
</tr>
<tr>
<td>07570100-A</td>
<td>0.40</td>
<td>0.40</td>
<td>0.38</td>
<td>0.42</td>
<td>0.38</td>
</tr>
<tr>
<td>07570100-F</td>
<td>0.40</td>
<td>0.43</td>
<td>0.37</td>
<td>0.43</td>
<td>0.41</td>
</tr>
<tr>
<td>07570120-A</td>
<td>0.40</td>
<td>0.41</td>
<td>0.41</td>
<td>0.42</td>
<td>0.42</td>
</tr>
<tr>
<td>07570120-A/F*</td>
<td>0.40</td>
<td>0.42</td>
<td>0.37</td>
<td>0.41</td>
<td>0.36</td>
</tr>
<tr>
<td>1007055-A</td>
<td>0.60</td>
<td>0.57</td>
<td>0.56</td>
<td>0.58</td>
<td>0.56</td>
</tr>
<tr>
<td>1007075-A/F*</td>
<td>0.60</td>
<td>0.56</td>
<td>0.53</td>
<td>0.56</td>
<td>0.52</td>
</tr>
<tr>
<td>1007085-A</td>
<td>0.60</td>
<td>0.54</td>
<td>0.51</td>
<td>0.52</td>
<td>0.51</td>
</tr>
<tr>
<td>10070100-A</td>
<td>0.60</td>
<td>0.55</td>
<td>0.53</td>
<td>0.55</td>
<td>0.54</td>
</tr>
<tr>
<td>10070100-F</td>
<td>0.60</td>
<td>n/a</td>
<td>n/a</td>
<td>n/a</td>
<td>n/a</td>
</tr>
<tr>
<td>10070120-A</td>
<td>0.60</td>
<td>0.54</td>
<td>0.54</td>
<td>0.56</td>
<td>0.54</td>
</tr>
<tr>
<td>10070120-F</td>
<td>0.60</td>
<td>0.51</td>
<td>0.50</td>
<td>0.50</td>
<td>0.49</td>
</tr>
<tr>
<td>1009055-A</td>
<td>0.70</td>
<td>0.59</td>
<td>0.58</td>
<td>0.60</td>
<td>0.60</td>
</tr>
<tr>
<td>1009075-F</td>
<td>0.70</td>
<td>0.65</td>
<td>0.62</td>
<td>0.64</td>
<td>0.62</td>
</tr>
<tr>
<td>10090100-F</td>
<td>0.70</td>
<td>0.68</td>
<td>0.64</td>
<td>0.67</td>
<td>0.65</td>
</tr>
<tr>
<td>10090120-F</td>
<td>0.70</td>
<td>0.65</td>
<td>0.63</td>
<td>0.63</td>
<td>0.60</td>
</tr>
</tbody>
</table>

6.3.3 Mechanical properties

Tensile testing of coupons was carried out to determine the mechanical properties of each material used in the test series, following the guidelines of the Australian Standard (AS 1391, 1991). Coupons for the base panel and the web material were directly obtained from the same coil used to manufacture the specimens in this series. The coupons of the top plate material were taken from the middle section of the flange between the longitudinal stiffeners after the cold-forming process. All materials were tested in the longitudinal, i.e. rolling direction which is also the principal loading direction of the actual panel in bending and testing of material transverse to the rolling direction was considered not to be necessary. The coated material of the base panel (G550) and the web (G350) was tested in its original coated state as well as in its uncoated state. However, the properties of the coated material with values based on the base metal thickness (b.m.t.) were then used for further calculations. A detailed description of the test method and all test results can be found in Appendix A. The average results for the different materials used in this test series are listed in Table 6-4.

Table 6-4. Average tensile coupon test results

<table>
<thead>
<tr>
<th>Coupon</th>
<th>Coating</th>
<th>$t_c$ (mm)</th>
<th>$t_b$ (mm)</th>
<th>$f_y^*$ (MPa)</th>
<th>$\varepsilon_{0.2}^*$ (%)</th>
<th>$f_u^*$ (MPa)</th>
<th>$E^*$ (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Base panel material</strong> (G550)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>G550/1.00-1</td>
<td>coated</td>
<td>1.07</td>
<td>0.99</td>
<td>677</td>
<td>0.54</td>
<td>687</td>
<td>232.9</td>
</tr>
<tr>
<td></td>
<td>uncoated</td>
<td>n/a</td>
<td>0.99</td>
<td>661</td>
<td>0.55</td>
<td>669</td>
<td>224.2</td>
</tr>
<tr>
<td>G550/0.75-1</td>
<td>coated</td>
<td>0.81</td>
<td>0.74</td>
<td>697</td>
<td>0.54</td>
<td>700</td>
<td>235.1</td>
</tr>
<tr>
<td></td>
<td>uncoated</td>
<td>n/a</td>
<td>0.74</td>
<td>695</td>
<td>0.55</td>
<td>696</td>
<td>226.0</td>
</tr>
<tr>
<td><strong>Web material</strong> (G350)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>G350/0.9</td>
<td>coated</td>
<td>0.94</td>
<td>0.89</td>
<td>391</td>
<td>0.43</td>
<td>487</td>
<td>224.0</td>
</tr>
<tr>
<td></td>
<td>uncoated</td>
<td>n/a</td>
<td>0.89</td>
<td>389</td>
<td>0.43</td>
<td>482</td>
<td>211.4</td>
</tr>
<tr>
<td>G350/0.7</td>
<td>coated</td>
<td>0.75</td>
<td>0.69</td>
<td>398</td>
<td>0.42</td>
<td>485</td>
<td>228.2</td>
</tr>
<tr>
<td></td>
<td>uncoated</td>
<td>n/a</td>
<td>0.69</td>
<td>382</td>
<td>0.44</td>
<td>469</td>
<td>206.9</td>
</tr>
<tr>
<td><strong>Top plate material</strong> (Blackform)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BF/2.5</td>
<td>uncoated</td>
<td>2.52</td>
<td>2.95</td>
<td>0.39</td>
<td>411</td>
<td>218.5</td>
<td></td>
</tr>
</tbody>
</table>

*$^*$ values based on b.m.t.
6.3.4 Modifications prior to testing

Further modifications of the panels were necessary prior to testing and were made at the Structural Engineering workshop of the University of Western Sydney, Australia.

- Two 20 mm reinforcing bars were stitch-welded to the top plate of each specimen, utilizing the natural groove of the longitudinal stiffeners of the top plate, as seen in Figure 6-7. This modification was crucial for the flexural tests to reduce the overall deflection of the panels, in particular when fracturing of the high-tensile base panel was expected. Increasing the section area of the top plate region had also a beneficial effect in shifting the centroid of the overall section towards the top plate, and therefore reducing the stress level of the top plate region, which consequently prevented a top plate buckling failure. The fillet welds (CIGWELD - FERROCRAFT 21, 3.2 mm) had a spacing similar to the TOX spacing in the middle region and a spacing approximately 1.5 times wider in the last third of the panel. The welds were alternated on each bar and the welding process started from midspan working simultaneously towards both panel ends to avoid locked-up stresses due to heat expansion of the material during the welding process. Although special care was taken, residual stresses remained in the base panel causing the section to bend. Table 6-5 gives an overview of the measured deflections (relative to the support points) and strains at midspan of each panel after the welding process, with strain gauge locations given in Figure 6-3.

Table 6-5. Measured deflections and strains at midspan after welding process

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Deflection (mm)</th>
<th>T-N (με)</th>
<th>N (με)</th>
<th>M (με)</th>
<th>S (με)</th>
<th>T-S (με)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0757060-A</td>
<td>35</td>
<td>96</td>
<td>20</td>
<td>n/a</td>
<td>67</td>
<td>30</td>
</tr>
<tr>
<td>0757080-A</td>
<td>44</td>
<td>108</td>
<td>43</td>
<td>89</td>
<td>9</td>
<td>67</td>
</tr>
<tr>
<td>07570100-A</td>
<td>25</td>
<td>119</td>
<td>54</td>
<td>132</td>
<td>22</td>
<td>89</td>
</tr>
<tr>
<td>07570120-A</td>
<td>28</td>
<td>58</td>
<td>3</td>
<td>71</td>
<td>5</td>
<td>71</td>
</tr>
<tr>
<td>0757080-F*</td>
<td>29</td>
<td>126</td>
<td>n/a</td>
<td>125</td>
<td>n/a</td>
<td>n/a</td>
</tr>
<tr>
<td>07570100-F</td>
<td>21</td>
<td>179</td>
<td>n/a</td>
<td>110</td>
<td>n/a</td>
<td>88</td>
</tr>
<tr>
<td>07570120-F*</td>
<td>30</td>
<td>78</td>
<td>n/a</td>
<td>112</td>
<td>n/a</td>
<td>103</td>
</tr>
<tr>
<td>1007055-A</td>
<td>34</td>
<td>97</td>
<td>29</td>
<td>101</td>
<td>38</td>
<td>105</td>
</tr>
<tr>
<td>1007085-A</td>
<td>25</td>
<td>107</td>
<td>16</td>
<td>94</td>
<td>38</td>
<td>119</td>
</tr>
<tr>
<td>10070100-A</td>
<td>21</td>
<td>69</td>
<td>40</td>
<td>67</td>
<td>43</td>
<td>97</td>
</tr>
<tr>
<td>10070120-A</td>
<td>28</td>
<td>78</td>
<td>n/a</td>
<td>138</td>
<td>n/a</td>
<td>103</td>
</tr>
<tr>
<td>1007075-F*</td>
<td>33</td>
<td>87</td>
<td>n/a</td>
<td>118</td>
<td>n/a</td>
<td>82</td>
</tr>
<tr>
<td>10070100-F</td>
<td>28</td>
<td>64</td>
<td>n/a</td>
<td>143</td>
<td>n/a</td>
<td>110</td>
</tr>
<tr>
<td>10070120-F</td>
<td>19</td>
<td>111</td>
<td>n/a</td>
<td>116</td>
<td>n/a</td>
<td>108</td>
</tr>
<tr>
<td>1009055-A</td>
<td>33</td>
<td>59</td>
<td>27</td>
<td>110</td>
<td>29</td>
<td>87</td>
</tr>
<tr>
<td>1009075-F</td>
<td>n/a</td>
<td>134</td>
<td>n/a</td>
<td>131</td>
<td>n/a</td>
<td>106</td>
</tr>
<tr>
<td>10090100-F</td>
<td>33</td>
<td>134</td>
<td>n/a</td>
<td>152</td>
<td>n/a</td>
<td>113</td>
</tr>
<tr>
<td>10090120-F</td>
<td>25</td>
<td>86</td>
<td>n/a</td>
<td>69</td>
<td>n/a</td>
<td>78</td>
</tr>
</tbody>
</table>

- 13 mm holes were drilled into the top plate in the constant moment region to vertically insert 12 mm reinforcing bars with pieces of Teflon glued to their ends which were then welded to the top plate (see on the left and middle of Figure 6-7). This alteration was deemed necessary to prevent base panel flange curling which forces the middle of the base pan to move towards the neutral axis of the section when the specimen exhibits large deflections. Flange curling occurs in flanges of thin-walled sections undergoing flexure and both tensile and compression flanges are subjected to it (Bernard et al., 1996; Yu, 2000). Curling of the base panel can cause significant
transverse bending within the closed section and relatively large bending moments to occur at the connections. Prying then causes additional tensile forces in the connections, resulting in a premature pull-out failure of the TOX connectors. The bars were usually 100 to 150 mm apart with the bar at midspan omitted which would otherwise influence the strain gauge measurements in this area.

- Preliminary testing revealed that a span of 5.9 m would have very large deflections and cause the TOX connectors in the base panel to fail in pull-out rather than shear. This was the main reason to reduce the span to 4.5 m resulting in different locations for load and support points. Since the jack load was applied through a distributed two-point load, only the support points were considered to be crucial in regard to a web buckling failure. Hence, 5 mm thick steel plates were screwed against the webs over the support and laterally held in place using prismatic steel blocks located between the plates and the rib of the base panel to avoid unwanted transverse movement of the web (see on the right of Figure 6-7). Both ribs of the base panel were then clamped to the plate between the base panel and the roller supports, simulating lateral restraint from adjacent panels in an actual application.

- Some of the specimen had to be transformed from configuration A into configuration F. This was achieved by using an oversized drill bit with a diameter of 12 mm which fully removed the unwanted TOX in the overhang of the base panel. To avoid separation of this section, the base panel was attached to the web by a loose clamp that allowed each part to move independently in longitudinal direction without longitudinal restraint.

![Figure 6-7. Stiffening elements at top plate (left and middle) and web (right)](image)

6.4 TEST RESULTS

6.4.1 General

In this test series, modifications were made to the specimens suppressing any unwanted failure modes, in particular related to the top plate, such as yielding, buckling or failure of the shear connectors between the top plate and the corrugated web. The following discussion is therefore relevant only to the longitudinal shear connection between the base panel and the corrugated web although similar shear forces exist in the longitudinal shear connection between the top plate and the corrugated web. However, this is of less interest since the connectors between the thick top plate and the webs are generally stronger compared to the connectors in the thinner base panel. However, the
general results of this study of shear connection performance in the base panel would be applicable to the shear connection in the top plate.

Table 6-6 contains ultimate load and ultimate moment results for all specimens. Deflections at ultimate load and failure mode are also included in the table. Deflections were measured at mid-span and did not include deflections due to the welding process (see Table 6-5) or the self-weight of the panel. Longitudinal slip measurements at ultimate load are summarized in Table 6-7. Maximum slip is stated together with the position of the LP as measured from the end of the panel. Individual test results, including vertical deflection, longitudinal slip and strain measurements, are contained in Appendix E.

### Table 6-6. Overview of test results

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Self-weight (kN/m)</th>
<th>Ultimate Jack Load (kN)</th>
<th>Ultimate Moment at midspan (kNm)</th>
<th>Deflection at midspan (mm)</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>0757060-A</td>
<td>0.1333</td>
<td>17.10</td>
<td>0.3000</td>
<td>16.03</td>
<td>1 / 25 fracture</td>
</tr>
<tr>
<td>0757080-A</td>
<td>0.1333</td>
<td>14.49</td>
<td>0.3000</td>
<td>13.58</td>
<td>1 / 47 shear</td>
</tr>
<tr>
<td>0757100-A</td>
<td>0.1333</td>
<td>12.88</td>
<td>0.3000</td>
<td>12.08</td>
<td>1 / 51 shear</td>
</tr>
<tr>
<td>0757120-A</td>
<td>0.1333</td>
<td>11.17</td>
<td>0.3000</td>
<td>10.47</td>
<td>1 / 62 shear</td>
</tr>
<tr>
<td>0757080-F†</td>
<td>0.1333</td>
<td>14.94</td>
<td>0.3000</td>
<td>14.01</td>
<td>1 / 43 shear</td>
</tr>
<tr>
<td>0757100-F</td>
<td>0.1333</td>
<td>12.34</td>
<td>0.3000</td>
<td>11.57</td>
<td>1 / 55 shear</td>
</tr>
<tr>
<td>0757120-F*</td>
<td>0.1333</td>
<td>10.52</td>
<td>0.3000</td>
<td>9.86</td>
<td>1 / 67 shear</td>
</tr>
<tr>
<td>1007055-A</td>
<td>0.1400</td>
<td>21.38</td>
<td>0.3150</td>
<td>20.04</td>
<td>1 / 22 fracture</td>
</tr>
<tr>
<td>1007085-A</td>
<td>0.1400</td>
<td>18.86</td>
<td>0.3150</td>
<td>17.81</td>
<td>1 / 35 shear</td>
</tr>
<tr>
<td>1007100-A</td>
<td>0.1400</td>
<td>17.26</td>
<td>0.3150</td>
<td>16.18</td>
<td>1 / 38 shear</td>
</tr>
<tr>
<td>1007120-A</td>
<td>0.1400</td>
<td>14.79</td>
<td>0.3150</td>
<td>13.87</td>
<td>1 / 44 shear</td>
</tr>
<tr>
<td>1007075-F†</td>
<td>0.1400</td>
<td>20.88</td>
<td>0.3150</td>
<td>19.58</td>
<td>1 / 28 shear</td>
</tr>
<tr>
<td>1007100-F</td>
<td>0.1400</td>
<td>16.56</td>
<td>0.3150</td>
<td>15.53</td>
<td>1 / 40 shear</td>
</tr>
<tr>
<td>1007120-F</td>
<td>0.1400</td>
<td>14.75</td>
<td>0.3150</td>
<td>13.78</td>
<td>1 / 45 shear</td>
</tr>
<tr>
<td>1009055-A</td>
<td>0.1433</td>
<td>22.27</td>
<td>0.3225</td>
<td>20.88</td>
<td>1 / 25 fracture</td>
</tr>
<tr>
<td>1009075-F</td>
<td>0.1433</td>
<td>20.40</td>
<td>0.3225</td>
<td>19.13</td>
<td>1 / 38 shear</td>
</tr>
<tr>
<td>1009100-F</td>
<td>0.1433</td>
<td>16.28</td>
<td>0.3225</td>
<td>15.26</td>
<td>1 / 54 shear</td>
</tr>
<tr>
<td>1009120-F</td>
<td>0.1433</td>
<td>13.89</td>
<td>0.3225</td>
<td>13.02</td>
<td>1 / 66 shear</td>
</tr>
<tr>
<td>1009055-F*</td>
<td>0.1433</td>
<td>22.27</td>
<td>0.3225</td>
<td>20.88</td>
<td>1 / 25 fracture</td>
</tr>
<tr>
<td>1009075-F</td>
<td>0.1433</td>
<td>20.40</td>
<td>0.3225</td>
<td>19.13</td>
<td>1 / 38 shear</td>
</tr>
<tr>
<td>1009100-F</td>
<td>0.1433</td>
<td>16.28</td>
<td>0.3225</td>
<td>15.26</td>
<td>1 / 54 shear</td>
</tr>
<tr>
<td>1009120-F</td>
<td>0.1433</td>
<td>13.89</td>
<td>0.3225</td>
<td>13.02</td>
<td>1 / 66 shear</td>
</tr>
<tr>
<td>† web buckling</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Table 6-7. Slip measurements at ultimate load

<table>
<thead>
<tr>
<th>Specimen</th>
<th>East-Side of Panel Max. slip (from panel end)</th>
<th>Slip at support (mm)</th>
<th>West-Side of Panel Max. slip (from panel end)</th>
<th>Slip at support (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0757060-A</td>
<td>0.554 (1830)</td>
<td>0.171</td>
<td>0.430 (2190)</td>
<td>0.009</td>
</tr>
<tr>
<td>0757080-A</td>
<td>0.595 (1830)</td>
<td>0.058</td>
<td>n/a</td>
<td>n/a</td>
</tr>
<tr>
<td>0757100-A</td>
<td>0.763 (1830)</td>
<td>0.115</td>
<td>0.863 (2190)</td>
<td>0.174</td>
</tr>
<tr>
<td>0757120-A</td>
<td>0.720 (2180)</td>
<td>0.247</td>
<td>0.699 (2190)</td>
<td>0.086</td>
</tr>
<tr>
<td>0757080-F†</td>
<td>0.869 (1920)</td>
<td>0.170</td>
<td>0.455 (1900)</td>
<td>0.022</td>
</tr>
<tr>
<td>0757100-F</td>
<td>0.748 (1940)</td>
<td>0.192</td>
<td>0.600 (1945)</td>
<td>0.065</td>
</tr>
<tr>
<td>0757120-F*</td>
<td>0.714 (1870)</td>
<td>0.162</td>
<td>0.401 (1505)</td>
<td>0.077</td>
</tr>
<tr>
<td>1007055-A</td>
<td>0.504 (1830)</td>
<td>0.109</td>
<td>0.672 (2190)</td>
<td>0.083</td>
</tr>
<tr>
<td>1007085-A</td>
<td>1.607 (2140)</td>
<td>0.355</td>
<td>2.060 (1820)</td>
<td>0.426</td>
</tr>
<tr>
<td>1007100-A</td>
<td>1.203 (1830)</td>
<td>0.089</td>
<td>1.273 (1830)</td>
<td>0.166</td>
</tr>
<tr>
<td>1007120-A</td>
<td>1.922 (1865)</td>
<td>0.410</td>
<td>1.829 (1840)</td>
<td>0.613</td>
</tr>
<tr>
<td>1007075-F†</td>
<td>1.150 (2275)</td>
<td>0.286</td>
<td>1.513 (1900)</td>
<td>0.331</td>
</tr>
<tr>
<td>1007100-F</td>
<td>1.854 (1930)</td>
<td>1.038</td>
<td>1.807 (1970)</td>
<td>0.703</td>
</tr>
<tr>
<td>1007120-F</td>
<td>1.446 (1870)</td>
<td>0.376</td>
<td>1.706 (1850)</td>
<td>0.888</td>
</tr>
<tr>
<td>1009055-A</td>
<td>0.244 (2065)</td>
<td>0.088</td>
<td>0.357 (2100)</td>
<td>0.056</td>
</tr>
<tr>
<td>1009075-F</td>
<td>0.425 (2255)</td>
<td>-0.095</td>
<td>0.584 (1670)</td>
<td>0.050</td>
</tr>
<tr>
<td>1009100-F</td>
<td>0.620 (1940)</td>
<td>0.096</td>
<td>0.579 (1970)</td>
<td>0.072</td>
</tr>
<tr>
<td>1009120-F</td>
<td>0.532 (2230)</td>
<td>0.135</td>
<td>0.577 (1850)</td>
<td>0.090</td>
</tr>
</tbody>
</table>
In general, the results for ultimate load in regards to TOX spacing show an approximate linear relationship for the specimen that experienced a shear failure of the connectors, independent of the material combination, as illustrated in Figure 6-8. Complete shear connection was possibly achieved at an approximate connector spacing of 65 mm for the specimen with a 1.00 mm base panel, and 60 mm for the specimen with a 0.75 mm base panel. Specimens with a spacing smaller than the critical value failed due to fracture of the base panel material. Specimens with a free end generally experienced a shear failure of all connectors whereas the connectors for specimen with end anchorage did not fail in the overhang, resulting in a web buckling near the last connector that failed in shear.

![Figure 6-8. Ultimate jack load results](image)

### 6.4.2 Complete shear connection

A state of complete shear connection can be assumed to have existed if a bending failure occurs at a critical cross-section and its moment capacity was not governed by the strength of the longitudinal shear connection (OneSteel Market Mills, 2001). This condition can be achieved when the number of connectors \( n \) in the length \( L_c \) between the support of the panel and the critical cross-section equals or exceeds the number required for complete shear connection, i.e. \( n \geq n_c \). Alternatively, this condition is achieved when the spacing \( s \) of the connectors is less than the critical spacing \( s_c \), where \( s_c = L_c / n_c \). As shown in Figure 6-8, it was possible to fracture a 1.00 mm base panel with a constant connector spacing of 55 mm, and a 0.75 mm base panel with a constant connector spacing of 60 mm. Considering the results for the 1.00 mm base panel, the critical spacing for complete shear connection would be about 65 mm. The fracture pattern of each specimen can be seen in Figure 6-9. Fracture was always of a brittle nature due to the low ductile behaviour of the high-tensile material used for the base panel, i.e. G550, and generally occurred at midspan. The fracture initiated at a single TOX location on one side only and slowly proceeded through the base panel to the other side where the fracture found its way to the closest TOX location before the loading process was finally stopped.
Although the fracture was of brittle nature, the overall ductility of the panels with complete shear connection was high, as can be seen in Figure 6-11. An indicator of the ductility of the panels is the ratio $\delta/L$ given in Table 6-6. All values are approximately four times higher than the limit of $1/100$.

Load-deflection curves of the specimens with a failure due to fracture of the base panel are presented in Figure 6-10. All specimens were cycled up to a load equivalent to 50% of the expected failure load for a couple of times before loading until failure was pursued. It was noted that the cycling process experienced a different load-deflection response compared to the initial loading, resulting in permanent deflections of the panels. Stress levels in the base panel, generated by the applied loads, where clearly below yield, and it was considered that the non-linear behaviour of the shear connectors was accountable for this. Once the LP’s measuring vertical deflection reached their limit of 100 mm, the test was put on hold to reset them. During this period, the applied load dropped slightly since all tests were carried out under position-control.

For all three specimens, large deflections were encountered before the specimens failed due to fracture of the base panel. In the case of specimen 1007055-A, one pair of loading fingers unintentionally slid off the loading bracket which made reloading necessary to complete the test. Prior to reloading, a permanent deflection of 85 mm was measured. Similar slopes were observed for the reloading process and the previous cycling process. Therefore, it is believed that the reloading process
did not significantly influence the ultimate load, but possibly reduced deflections when fracture occurred. Figure 6-11 pictures the final stage of the loading process of specimen 1009055-A. The deflections at ultimate load were approximately $L/25$, i.e. 4% of the span $L$.

Each specimen had five resistance strain gauges attached to the bottom of the base panel at mid-span, as indicated in Figure 6-3. Graphs of all the strain readings are contained in Appendix E. The strain measurements were then transformed into equivalent stresses, using an idealized material curve which was based on a modified Ramberg-Osgood curve. For materials without a distinct yield plateau, typical for nonlinear materials such as G550, the Ramberg-Osgood curve (Ramberg and Osgood, 1943) gives good results for the relationship between stress and strain of the actual material and is defined by three parameters, viz. the initial elastic modulus ($E_0$), a proof stress, and a positive constant ($n$) that defines the sharpness of the knee of the stress-strain curve. In the absence of a distinct yield plateau, it is common practice to define an equivalent yield stress, which is usually chosen as the stress with a plastic strain of 0.2%, called the 0.2% proof stress ($\sigma_{0.2}$). The modified Ramberg-Osgood curve is expressed in the following form if the proof stress is chosen as the 0.2% proof stress (Bezkorovainy et al., 2003).

$$\varepsilon = \frac{\sigma}{E_0} + 0.002 \left( \frac{\sigma}{\sigma_{0.2}} \right)^n$$  \hspace{1cm} (6-1)

The values of the quantities ($E_0$, $\sigma_{0.2}$, $n$) are given in Table 6-8 for the 1.00 mm and 0.75 mm base panel material.

<table>
<thead>
<tr>
<th>Material</th>
<th>$E_0$ (GPa)</th>
<th>$\sigma_{0.2}$ (MPa)</th>
<th>$n$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.00 mm (G550), coated</td>
<td>233</td>
<td>677</td>
<td>45</td>
</tr>
<tr>
<td>0.75 mm (G550), coated</td>
<td>235</td>
<td>697</td>
<td>20</td>
</tr>
</tbody>
</table>
A typical load-stress curve based on strain gauge measurements is presented in Figure 6-12 for specimen 1009055-A. The strain gauges located furthest away from the neutral axis (T-S, T-N and M) show the highest stress levels, whereas the other strain gauges (S and N) are closer to the neutral axis and show lower stresses, as expected, due to the strain gradient. Local effects and flange curling are believed to be the major contributors for the increased stresses in the middle pan (M). It can be seen that at the ultimate load (22.27 kN) when fracture occurred, the stresses at the T-S, T-N and M locations reached the ultimate tensile capacity of the sheeting material (687 MPa). This confirms the results of the tests in Chapter 3.2.5, where it was shown that a presence of a small TOX in a wide panel would have an insignificant effect on the tensile strength of the panel, even in G550 steel.

![Figure 6-12. Load-stress curve of specimen 1009055-A](image)

### 6.4.3 Partial shear connection

If the nominal moment capacity is governed by the strength of the connections between the base panel and the corrugated web (unwanted failure modes related to the top plate were suppressed), the critical cross-section can be deemed to exhibit partial shear connection, i.e. $n < n_c$ (OneSteel Market Mills, 2001). Depending on the spacing and strength of the press-joints, different degrees of shear connection ($\beta = n/n_c = s/s_c$) can be exhibited, noting that the strength of the press-joints is essentially a function of the material properties and thicknesses of the corrugated web and base panel (Chapter 3.1.5). Figure 6-13 shows the test results for the 1.00 mm and 0.75 mm base panel where a value of $\beta$ greater than one indicates complete shear connection. A value of $\beta$ equal to zero indicates no shear connection at all, and the member components would act individually with virtually zero moment capacity for the hybrid deck. This effect is indicated by the dashed line and differs from a typical composite steel-concrete beam where the steel section does have moment capacity without connection to the concrete (OneSteel Market Mills, 2001).
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Degree of shear connection \( \beta \)

\[ \frac{M}{M_0} \]

Complete shear connection

Partial shear connection

Figure 6-13. The effect of degree of shear connection on moment capacity

In the following, specimen 10070100-A (with end-anchorage) and 10070100-F (with free end) were selected as representative examples of all other tested specimens with shear failure of the TOX connectors and are discussed in more detail. Individual test results of all specimens can be found in Appendix E. As a result of the test configuration, different failure modes in shear were detected, as indicated in Table 6-6. In configuration A, connectors in the overhang and near the support did not fail, causing the lower web flanges to buckle near the last connector that failed in shear, as seen in Figure 6-14. It appears that longitudinal slip was not transferred to the end of the panel and the connectors past the shear span, i.e. the overhang, acted like an anchorage system. In contrast, all connectors in configuration F, with connectors in the overhang being removed prior testing, experienced a shear failure, allowing the accumulated slip to be transferred up to the ends, as seen in Figure 6-14.

Figure 6-14. Failure modes for partial shear connection
Figure 6-15 compares the load-deflection curves of the two specimens. The specimens were cycled up to a load equivalent to 50% of the expected failure load for a couple of times before loading until failure was pursued. The overall stiffness of both panels was similar up to a load of around 11 kN, and started to differ slightly at higher load levels, with configuration A being stiffer and therefore deflecting less. Also the maximum load was 4% higher than in configuration F.

![Figure 6-15. Load-deflection curve of specimen 10070100-A and -F](image)

A closer look at the longitudinal slip measurements (taken on the north side of the panel only as seen in Figure 6-3 but it is believed that similar slip values were developed on the south side) reveals that specimen 10070100-F (Figure 6-17) experienced higher slip values along the shear span (the location of each measurement point is stated in the boxes on the graph and defines the distance from the end of the panel in millimetres) than its counterpart with end anchorage (Figure 6-16) and, consequently, is the reason for the increase in deflection. The measurements for specimen 10070100-A show almost no slip in the overhang region (0 to 750 mm from panel end) compared to configuration F with relatively high slip values even over the support (750 mm from panel end). A linear relationship between load and slip can be observed for loads up to about 11 kN, with slips rapidly increasing for loads beyond this value, depending on the position of the slip measurement. The lowest slip values are in the region of the supports whereas the highest values are located towards the applied load (around 1900 mm from panel end). Eventually the connector with the highest slip fails in shear, leading to failure of the shear connection. Higher slip values were measured in the specimen with a free end as it allows a greater degree of redistribution of slip along the length of the panel compared to the specimen with the ends anchored. It is believed that the redistribution of slip in this redundant system allows some connectors to exceed their slip capacity at ultimate load and deform into the post-ultimate range. The post-ultimate load-slip response of individual connectors can be seen in Chapter 3.1.5.
When the test was put on hold for about two to three minutes to reset the 100 mm LP’s, the applied load (15.5 kN and 14 kN, respectively) dropped by 0.3 kN during that period. Although the deflection of the specimen did not change at all, as can be seen in Figure 6-15, slip of the more severely deformed press-joints continued to grow. This behaviour can be described as creep which is a time-dependant increase in deformation due to sustained constant load.

**Figure 6-16. Slip measurements on east-north side of Specimen 10070100-A**

**Figure 6-17. Slip measurements on east-north side of Specimen 10070100-F**

Figure 6-18 and Figure 6-20 display the slip distribution along the east side of the panels at certain applied loads (stated above each line in kilo-Newton) with data extracted from the above slip
measurements. In configuration A (Figure 6-18), slip values are still quite moderate for loads up to 11 kN and show an approximate linear distribution between the support and the last measurement towards the applied load (2190 mm from panel end) with values around 0.2 mm. Slip values over the support and in the overhang are virtually zero due to the anchorage at the overhang. From 11 kN on, the shape of the slip distribution curves are alike with maximum slip values rapidly increasing at higher loads. At failure, maximum slip (about 1.2 mm) was measured in the last third of the shear span whereas slip in the overhang was still zero. This behaviour is similar to the results of experiments on steel-concrete composite beams (Figure 6-19) where the shear connection continued beyond the support and a point load was applied at midspan (Chapman and Balakrishnan, 1964). Since only two connectors were located in the overhang of specimen E1, slight propagation of longitudinal slip could still occur in the overhang. Even though some slip was measured over the support, the effect of anchorage is obvious. The large slip at the quarter points was considered to be due to the high interface shear in the plastic region.

![Figure 6-18. Slip distribution on east-north side of specimen 10070100-A](image)

![Figure 6-19. Slip distribution of beam with overhang (Chapman and Balakrishnan, 1964)](image)
In configuration F (Figure 6-20), the slip distribution is essentially different although slip values for loads up to 11 kN are also moderate and show a similar linear distribution. However, the most remarkable difference is that slip occurred over the supports at higher loads with values equal to or less than 50% of the maximum slip measured at a certain load. Again, the shape of the curves is similar for loads more than 11 kN and maximum slip is localized to a small region about two thirds (1930 mm from panel end) away from the supports. This phenomenon is in stark contrast to the slip distribution obtained by the classic elastic Newmark analysis (Newmark et al., 1951; Siess et al., 1952) where maximum slip occurs at the free end, and is discussed later in Chapter 6.7.2. However, the phenomenon of maximum slip away from the free end has been observed in tests on composite steel and lightweight concrete beams loaded with a single point load at midspan (Roderick et al., 1967) as shown in Figure 6-21. Maximum slip was measured at the centre of the shear span for loads less than the concrete slab cracking value, i.e. 27 kips, and the shape of the slip distribution curve changes rapidly after the cracking occurred, with maximum slips at the quarter points (cracked area).
For both specimens, a rapid increase of slip is noticeable at higher loads. In Figure 6-22, maximum slip values within the shear span are plotted against the jack load for specimens with different connector spacings which are indicated on the individual curves on the graph. The slip development of all tested specimens shows an exponential behaviour with increasing loads and asymptotes to the vertical at failure. Specimens with 0.90 mm webs show less slip at the same load compared to similar specimens with 0.70 mm webs which is consistent with the observed pure shear behaviour of individual TOX connectors (Chapter 3.1.5 and Chapter 5.4.3).

![Figure 6-22. Development of maximum slip on east-north side](image)

### 6.5 DETERMINATION OF CONNECTOR FORCES

In a composite steel-concrete beam, the strength and deformation capacity of the shear connection can be estimated from a load-slip curve that relates shear force per connector and longitudinal slip (Patrick et al., 1994). Using longitudinal slip measurements of the tested specimens and load-slip curves of individual connectors, it is possible to calculate approximate connector forces in the vicinity of where the measurement was taken. The behaviour of the connector in pure shear for each material combination can be established based on the results obtained from shear tests on single connectors as described in Chapter 3.1.5, or from tests performed on the actual section as discussed in Chapter 5.4.3. The non-linear behaviour can be simplified into a tri-linear load-slip relationship providing three different connector stiffnesses ($k_1$, $k_2$, and $k_3$) which describes the connector force up to a certain slip value. As an example, the idealized connector load-slip curve of material combination 10070 in pure shear is shown in Figure 6-23 based on the results obtained from individual shear tests (B18-10070-3), and tests performed on the actual section (1007090-2) which contained two connectors. Normalizing the ultimate load was therefore necessary to compare the two different testing methods whereas the development of slip was considered to be similar for one or two connectors. It can be seen that the two test methods give similar results although slightly higher initial stiffnesses were measured for the tests performed on the actual section. This was possibly a result of how the slip measurements were obtained (refer to Chapter 5.2.3). For this reason, stiffness $k_1$ of the idealized load-slip curve was based on the behaviour of the individual shear tests rather than the mean of both testing methods. The idealized model only considers slip up to the maximum load and does not include the post-ultimate load-slip response.
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Figure 6-23. Connector load-slip curves of material combination 10070 in pure shear

Similar idealized connector load-slip curves can be established for the other material combinations used in this test series, and values for their connector stiffness and range of validity are presented in Table 6-9. Values of the stiffness represent two connectors (the TOX located on the north and on the south side of the panel) and were calibrated to the ultimate shear capacity achieved with the tests performed on the actual section since only those were made with the TOX machine used in this test series, noting that the shear capacities of the two testing methods are similar. However, it should be also mentioned that the idealized curve was established on test results using slightly different materials than used in this test series, but the differences in the mechanical properties was considered to be negligible for the purpose of this investigation.

<table>
<thead>
<tr>
<th>Material combination</th>
<th>Max. Load (kN)</th>
<th>$k_1$ (N/mm)</th>
<th>Max. slip (mm)</th>
<th>$k_2$ (N/mm)</th>
<th>Max. slip (mm)</th>
<th>$k_3$ (N/mm)</th>
<th>Max. slip (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0757090</td>
<td>6.59</td>
<td>70,500</td>
<td>0.07</td>
<td>8,800</td>
<td>0.22</td>
<td>900</td>
<td>0.60</td>
</tr>
<tr>
<td>1007090</td>
<td>8.29</td>
<td>62,200</td>
<td>0.10</td>
<td>4,700</td>
<td>0.45</td>
<td>500</td>
<td>1.33</td>
</tr>
<tr>
<td>1009090</td>
<td>7.69</td>
<td>61,500</td>
<td>0.10</td>
<td>6,900</td>
<td>0.30</td>
<td>600</td>
<td>0.58</td>
</tr>
</tbody>
</table>

The connector force distribution along the east side of the panels at certain applied loads is presented in Figure 6-24 (specimen 10070100-A) and Figure 6-25 (specimen 10070100-F). It is quite remarkable that at a jack load of only 4 kN, the connectors close to the loading points already reach approximately half of their maximum load carrying capacity. A similar shape of connector force distribution is maintained for jack loads up to about 7 kN. This is the point when the connectors close to the loading points start to lose their initial stiffness $k_1$ (see Figure 6-23). At a jack load of around 11 kN, the majority of the connectors in the shear span of the specimen with free ends reach 80% of their maximum load carrying capacity. This is not possible for the specimen with end anchorage where the connectors in the overhang prevent the development of slip, thus also the development of
connector forces. Only the connectors furthest away from the overhang, i.e. close to the loading points, are able to develop connector forces of a similar load level. A further increase of the jack load results in higher connector forces along the shear span. However, the increase in connector force occurs at a considerably slower pace compared to the increase of slip due to the major loss in connector stiffness. Nearing ultimate jack load at the panel, all (for specimen 10070100-F) and nearly all (for 10070100-A) connectors in the shear span are close to their maximum load capacity showing a uniform distribution of connector shear force over the shear span.

![Figure 6-24. Connector shear force distribution on east side of specimen 10070100-A](image)

![Figure 6-25. Connector shear force distribution on east side of specimen 10070100-F](image)
From Figure 6-24, it becomes evident that close to failure, the connectors in the overhang do not reach their maximum load capacity thus cannot be fully mobilised. For design purposes, it would be more conservative to ignore the connectors in the overhang and only make use of the connectors in the shear span. Eventually, the connector with the highest slip will fail in shear. The applied jack load has now to be carried by fewer connectors, and the total longitudinal shear force is quickly redistributed to the remaining connectors. The connector adjacent to the failed connector will very soon also fail in shear, creating a form of chain reaction which “unzips” most of the connectors in the shear span.

### 6.6 INVESTIGATION OF A FAULTY TOX JOINT

On very rare occasions, it might happen that the clinch process of a TOX joint cannot be completed (e.g. the penetration depth of the punch tool is less than required due to a pressure drop, a damaged/broken tool or other physical obstruction) and a faulty TOX joint is produced (Figure 6-26).

![Figure 6-26. Faulty TOX joint in the base panel (enlarged on the right)](image)

During manufacturing, such an occasion would be generally indicated by the TOX machine and its operator and this allows placing additional joints adjacent to the faulty joint. However, it was considered to be prudent to investigate the effect of an undetected, faulty TOX joint on the overall performance of the hybrid steel deck. The faulty joint was simulated by completely removing the TOX with a 12 mm drill bit, thus removing its ability to carry any longitudinal slip (see Figure 6-27). To create a symmetric situation allowing proper measurement of behaviour, one TOX was removed on either side of the panel (north and south side). It was decided to have the faulty joints placed in a region of high longitudinal slip demand which was believed to give the greatest impact on the load carrying capacity of the panel. The TOX were therefore removed approximately 1,800 mm away from the east end of the panel, as seen in Figure 6-28.

![Figure 6-27. Removed TOX joint and measurement of slip adjacent to missing TOX](image)
Besides the standard instrumentation of 100 mm LP’s, 15 mm LP’s and strain gauges (refer to Chapter 6.2.3), additional 15 mm LP’s were placed on the north side between individual TOX adjacent to the missing TOX (Figure 6-27) to closely monitor the slip development around the discontinuity. The panel was of configuration F (free end) with the connectors in the overhang being removed. The following tables give additional information about the tested panel such as number of connectors in the shear span/overhang (Table 6-10), the average bottom thickness of the TOX (Table 6-11) and the measured deflection and strains at midspan after the 20 mm reinforcement bars were welded onto the top plate (Table 6-12).

**Table 6-10. Number of connectors**

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Spacing (mm)</th>
<th>East-North</th>
<th>East-South</th>
<th>West-North</th>
<th>West-South</th>
</tr>
</thead>
<tbody>
<tr>
<td>10090100-F (missing TOX)</td>
<td>100</td>
<td>16 / 1</td>
<td>16 / 1</td>
<td>17 / 2</td>
<td>17 / 1</td>
</tr>
</tbody>
</table>

**Table 6-11. Average bottom thickness of TOX**

<table>
<thead>
<tr>
<th>Specimen</th>
<th>East-North (mm)</th>
<th>East-South (mm)</th>
<th>West-North (mm)</th>
<th>West-South (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10090100-F (missing TOX)</td>
<td>0.65</td>
<td>0.57</td>
<td>0.60</td>
<td>0.57</td>
</tr>
</tbody>
</table>

**Table 6-12. Measured deflection and strains at midspan after welding process**

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Deflection (mm)</th>
<th>T-N (με)</th>
<th>N (με)</th>
<th>M (με)</th>
<th>S (με)</th>
<th>T-S (με)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10090100-F (missing TOX)</td>
<td>25</td>
<td>82</td>
<td>n.a.</td>
<td>91</td>
<td>n.a.</td>
<td>101</td>
</tr>
</tbody>
</table>

At ultimate load (14.94 kN), the panel failed on the side containing the missing TOX (east side) by shearing off all TOX connectors in the shear span and in the overhang, whereas none of the connectors on the west side failed. Removing one pair of TOX joints in a critical area of the shear span can have a significant impact on the load carrying capacity of the panel and reduce the ultimate load by approximately 8 % compared to a similar panel without imperfections (Specimen 10090100-F). The missing TOX also affected the overall deflection at midspan which was increased by 5 % at a similar load level. Individual test results, including lateral deflection, longitudinal slip and strain measurements, are contained in Appendix E. Figure 6-29 and Figure 6-30 show the characteristic longitudinal slip distribution of the base panel connection on the east and west side, respectively.
It can be seen, that the missing TOX had a significant effect on the slip distribution of the panel. The TOX between the loading point and the discontinuity experienced excessive slip values at ultimate load (around 0.8 mm), and dropped considerably (to about 0.45 mm) just beyond the missing TOX. Slip values then slowly decreased towards the support. On the west side of the panel, slips did not surpass 0.3 mm, with values close to zero near or at the support. The missing TOX led to a redistribution of slip, resulting in considerably higher slips in an adjacent TOX compared to that in a similar panel without the imperfection (10090100-F) at the same load, and this resulted in a premature
failure of the panel. It was noted that the removal of the TOX led to an asymmetric distribution of slip with increased slips on the side with the missing TOX (Figure 6-29) and decreased slips on the other side (Figure 6-30). In the specimen without the imperfection, the slip distribution was generally similar on both sides.

The results of the tests indicate that maintaining a regular connector spacing is essential to the proper performance of the hybrid steel deck.

6.7 MODELLING THE BEHAVIOUR

6.7.1 General

Static forces and their distribution for composite structures with rigid connections can be obtained by a linear elastic analysis and is often used in practice where longitudinal shear forces are proportional to the vertical shear forces. For composite structures with semi-rigid connections, this proportionality does not apply any more and the static forces and their distribution are not well known. The determination of the longitudinal shear forces is then influenced by various factors, mainly the stiffness of the shear connectors. Considering a beam with single span and a two-point central load, the linear elastic analysis assumes no longitudinal slip in the constant moment region which is only valid for rigid connections. Semi-rigid connections on the other hand have an additional statically indeterminate effect on the composite member. To account for the continuity of deformation, longitudinal slip propagates up to midspan, generating longitudinal shear forces even in the constant moment region between the loads (Ramm et al., 1999). The determination of a closed-form solution for a member with semi-rigid longitudinal shear connection requires the development of a model that is a function of the bending moment. However, static forces for indeterminate systems are dependent on the overall stiffness of the entire system and, in particular for semi-rigid connections, the stiffness is not constant over the length of the beam. Therefore, calculations by hand are not practical and computational models are required to overcome this problem.

6.7.2 Computational model

A computational model using the finite element method that takes account of the flexibility of the shear connectors would be a state-of-the-art solution for this highly non-linear problem, but also very time consuming to develop. For this reason a simpler computational model, based on a Vierendeel truss analogue, was generated for simulating deformation as well as slip of the tested specimens, and to demonstrate the soundness of the assumptions. Models with a similar approach have already been used successfully in multi-layered timber structures with semi-rigid connections (Gliniorza et al., 2002). A Vierendeel truss has continuous upper and lower members, connected by vertical web members with rigid joints. This statically indeterminate truss has no diagonal members and all members are subject to bending moments, i.e. flexural members. Results for Vierendeel truss analogues are very close to those obtained with classical formulae in statically defined systems (Ceccotti, 2002). The commercial software package MICROSTRAN (MICROSTRAN V7.0, 1999) was used to built the model. It is normally used to analyse and design 2-D and 3-D frames and trusses,
so individual sections of the panel were represented by beams. The model was designed to simulate the behaviour of the shear connectors between the base panel and the web only and did not allow for slip in the shear connection between the top plate and the web by assigning rigid connections between the latter. This simplification was justified by the fact that additional self-piercing rivets were placed between the generally stronger TOX in the top plate, and the fact that two 20 mm reinforcing bars were attached to the top plate, reducing the stress level of the top plate region and consequently the longitudinal shear forces on the connectors in the top plate. Figure 6-31 shows one half of the computational model for specimen 10070100-A.

The upper beam section had the same cross-sectional area and second moment of area as the 2.5 mm top plate with the 20 mm reinforcing bars attached to it. The lower beam had the same cross-sectional area and second moment of area as the base panel section and the distance \( y_c \) between the two beams was equal to the distance between the centroids of the upper and lower beam element sections. Vertical beam members, with the length of the actual web height \( y_w \), replaced both webs and were placed at the same spacing as the TOX connectors. The second moment of area of the web member section \( I_w \) was set equal to that of a strip of web with a depth equal to the spacing of the connectors and a width equal to the combined effective thickness of the webs allowing for the web inclination in the actual cellular deck. The effect of the corrugated web was considered in the analysis (Papangelis and Hancock, 1999). This web assumption was considered adequate as a sensitivity analysis showed that variations in stiffness of the web at this level of web stiffness had little influence on the overall deflection of the truss. It should be noted that while the stiff vertical web members allow the transfer of longitudinal shear between the upper and lower beam members, they do not include the in-plane bending stiffness of an actual continuous web member with a distribution of longitudinal normal stress from compression at the top to tension at the bottom. Hence, the overall stiffness of the Vierendeel truss is less than that of the hybrid deck with a continuous web.

In contrast to the classical Vierendeel truss with rigid connections, additional short vertical members are used to model the slip behaviour of the TOX connectors in the base panel. The members are rigidly connected to the end of the web-beam and the lower beam, having a distinct length \( l \), which is the difference between \( y_w \) and \( y_c \). The stiffness \( I_T \) of the connector-member is based on the stiffness \( k \) obtained from the load-slip curve of the connector (see Chapter 6.5), and has to be adjusted...
to match the length of the member in the model. The following equation, originated from a beam with both ends clamped, was used to determine the equivalent TOX second moment of area $I_T$:

$$k = \frac{P}{\delta} = \frac{12EI}{l^4} \Rightarrow I_T = \frac{kl^4}{12E}$$

(6-2)

Since the load-slip behaviour of each connector is highly non-linear, the behaviour of the specimen can be simulated by a linear-elastic analysis using a staged iterative calculating process. The analysis can be further simplified by using an idealized load-slip curve that limits the change of stiffnesses of the connector. The values for a tri-linear model are given in Table 6-9 and can be used in an iterative analysis using the elastic computational model in the following manner.

The process starts with the initial connector stiffness $k_1$ for all connectors. Applying load to the system will introduce longitudinal shear forces into the connectors. The load can be increased (in incremental steps) until the first connector reaches the maximum slip value assigned to the stiffness $k_1$. The stiffness of this particular connector changes then to $k_2$ whereas all other connectors still use stiffness $k_1$. The load can now be further increased until the next connector reaches the maximum slip value and changes stiffness. This procedure can be repeated as more connectors change their stiffness from $k_1$ to $k_2$. Continuing to increase the load will result in higher longitudinal shear forces and therefore also higher slip values. Once one connector reaches the maximum slip value assigned to stiffness $k_2$, the stiffness will change to $k_3$. This, of course, might already happen before all connectors have changed their stiffness from $k_1$ to $k_2$, but would be taken into account during the iterative calculating process. A further increase of load will make the next connector change its stiffness to $k_3$.

Nearing ultimate load of the system, nearly all connectors in the shear span will have changed their stiffness from $k_2$ to $k_3$. This is also indicated by the uniform distribution of connector shear force over the shear span where the connectors are close to their maximum load capacity, as can be seen in Figure 6-24 and Figure 6-25. At ultimate load, the connector with the highest slip will fail in shear. Redistribution of the longitudinal shear loads will result in failure of the adjacent connector with the highest slip. This mechanism will continue with the failure of a number of connectors in the shear span until a state of equilibrium between load and remaining connectors has been reached.

To obtain values for the ultimate failure load, longitudinal slip, as well as the overall deflection of the system, individual results for load, deflection and slip must be recorded for each load increment that occurred during the iterative calculation process and added together to gain the final result. This can be a cumbersome procedure and automation is required to process the huge amount of data which was not possible in the time frame of this thesis. However, to demonstrate the soundness of the model, “snapshots” of the behaviour can be taken at certain loading stages. It is known, that at the beginning of the loading process, all connectors have the same connector stiffness $k_1$, and the slip distribution obtained during the analysis is representative for slip values equal to or less than the maximum value assigned to the stiffness $k_1$. At ultimate load, it can be assumed that most of the connectors have a stiffness of $k_3$ and the first connector to fail has reached its maximum slip capacity. Similarly, it can be assumed that most of the connectors have reached the stiffness $k_2$ when the first connector changes its
are shown in Figure 6-32 for specimen 10070100-A and Figure 6-33 for specimen 10070100-F.
The curve with the connector stiffness $k_1$ shows a similar slip distribution as described in Newmark’s analysis of composite beams with incomplete interaction (Newmark et al., 1951), with slips increasing along the shear span and maximum values over or near the supports. The distribution is only appropriate for slips up to 0.1 mm and describes the behaviour of the initially stiff connectors. Once the connectors have lost their initial stiffness, the shape of the slip distribution curve differs considerably from Newmark’s theory and would look similar to the curve with the connector stiffness $k_2$ where maximum slip occurs in the middle of the shear span and drops over the supports. The curve with the connector stiffness $k_3$ shows the slip distribution when all connectors have the same stiffness $k_3$ and the first connector has reached its maximum slip capacity. Comparing this slip distribution with the actual slip distribution at maximum load already shows similar characteristics with maximum slip occurring within the shear span and slips diminishing towards the end of the specimen.

It is apparent that the slip distribution is quite sensitive for connectors with a very low stiffness since only a small increase in longitudinal shear force will result in large amounts of slip. As already mentioned earlier, not all the connectors, in particular those in the overhang of the specimen, had slipped sufficiently at ultimate load to be considered to have stiffness $k_3$. Therefore, the analysis for the specimen with overhang (10070100-A) was re-run using connector stiffnesses more appropriate to the actual slips, viz. $k_2$ in the overhang and $k_3$ in the shear span. The results are shown in Figure 6-34 where it can be seen that the agreement between actual and analytical slip distribution is improved.

For the specimen with a free end (10070100-F), a similar improvement in the agreement between the actual and analytical slip distribution can be seen by comparing Figure 6-33 and Figure 6-35, resulting from the use of a higher connector stiffness for the last connector over the support. This demonstrates the sensitivity of the load-slip distribution to the connector stiffness, even for one
Chapter 6. Base panel shear tests

It should be noted that in the actual test, some restraint to slip is provided at the support through friction. This is ignored in the analytical model.

![Figure 6-35. Slip distribution of actual specimen and modified model (10070100-F)](image)

As already shown in Figure 6-15, the load-deflection relationship for the panel is essentially linear up to a jack load of about 11 kN. With increasing load, the deflections become larger at a greater rate than the applied load. This is considered to be mainly due to the non-linear behaviour of the load-slip characteristics of the TOX connector, as shown in Figure 6-23. The load-deflection curve of the panel can be determined by a linear-elastic analysis using a staged iterative calculating process and the computational truss model as described above. Similar to the “snapshots” taken for the slip distribution of the panel, the slope of this curve can be calculated at certain loading stages where the stiffness of the individual connectors is approximately known. This slope is called the tangent stiffness and is defined as the quotient of the load increment and the increment of deflection. At the beginning of the loading process, all connectors have the same connector stiffness $k_1$. Using this stiffness in the computational model for all connectors, the load-deflection response of the panel is similar to the actual load deflection response of the panel (see Figure 6-36 for specimen 10070100-A, and Figure 6-37 for specimen 10070100-F), although slightly underestimating the overall stiffness of the panel. This can be explained by the fact that the panel is modelled as a Vierendeel truss which does not include the in-plane bending stiffness of an actual continuous web member. However, the difference in overall stiffness of the panel is less than 5%. Calculating the linear-elastic deflection of the panel based on section properties, assuming the web is fully effective and no slip between the base panel and the web occurs, overestimates the overall stiffness as would be expected.

At ultimate load (17.26 kN for specimen 10070100-A and 16.56 kN for 10070100-F), it can be assumed that most of the connectors have reached connector stiffness $k_3$. Using this stiffness ($k_3$) in the model, the tangent stiffness at this point can be determined by dividing the obtained deflection by
the applied load. The results show good agreement with the actual load-deformation response at maximum load. From the load-slip curve for the connector, the stiffness $k_2$ is tangent to the curve at a slip of approximately 0.28 mm. Values of slip of this order correspond to a jack load of about 12 kN for specimen 10070100-F and of about 13 kN for specimen 10070100-A. Using this stiffness ($k_3$) in the model, the tangent stiffness at this point can be determined by dividing the obtained deflection by the applied load. The results show good agreement with the actual load-deformation response at these levels of loads.

**Figure 6-36. Tangent modulus at selected points of specimen 10070100-A**

![Tangent modulus at selected points of specimen 10070100-A](image1)

**Figure 6-37. Tangent modulus at selected points of specimen 10070100-F**

![Tangent modulus at selected points of specimen 10070100-F](image2)
The results demonstrate the soundness of the model and clearly indicate that the non-linear load-deflection response is effectively due to the non-linear behaviour of the shear-connection of the TOX.

6.7.3 Simplified physical models

One of the main purposes of the TOX connectors in the hybrid steel deck is to resist longitudinal slip. Assuming no shear connection exists between the individual components, longitudinal slip can occur freely and each component acts separately. Much greater bending strength and stiffness can be achieved if the components interact by using shear connectors to limit longitudinal slip. Figure 6-38 illustrates the simply-supported hybrid steel deck, assuming a rigid link between the top plate and the web and TOX connectors located only between the base panel and the web. This simplification is justified for this series of tests where additional self-piercing rivets were placed between the generally stronger TOX in the top plate and two 20 mm reinforcing bars were attached to the top plate. The hybrid steel deck is subjected to a concentrated load at midspan and consequently bends developing compressive forces $C$ in the top plate region, tensile forces $T$ in the base panel region and longitudinal shear forces in the interface between base panel and web (see free-body diagram in Figure 6-38).

![Figure 6-38. Shear connection resisting longitudinal shear](image)

Tests have shown that the distribution of connector forces can be considered as uniform over the shear span at the maximum load on the panels (see Chapter 6.5) and that the connector force can be taken as the shear capacity of the connector. Therefore, for design purposes, the load-slip curve of the TOX connectors can be idealized as rigid-plastic behaviour whereby the shear capacity at each connector $F_{\text{TOX}}$ is assumed to be constant irrespective of the amount of slip. This assumption, also used in the partial shear connection strength theory for composite beams of the Australian Standard (AS 2327-1, 2003), can be adopted for the design of these panels. Therefore, at the critical cross-section in the panels (loading point near midspan in the tests), the resultant tensile force in the base panel $T_{\text{BP}}$ is the sum of the connector forces

$$T_{\text{BP}} = n F_{\text{TOX}}$$

(6-3)

where $n$ is the number of connectors from the critical cross-section to the support of the panel. For panels with end anchorage, it is more conservative to ignore the connectors in the overhang since they do not reach their maximum load capacity and thus cannot be fully mobilised (see Figure 6-24). Values for the connector force $F_{\text{TOX}}$ of different material combinations were obtained from Table 6-9. It should be noted that friction between the web and the base panels has been conservatively ignored in the model as was any component of longitudinal force developed in the panel arising from restraint.
at the supports. This latter restraint has been shown to have a significant effect on composite decks (Patrick, 1994) but no attempt has been made to quantify it in these tests. However, the resultant tensile force in the base panel cannot exceed the tensile capacity of the base panel at fracture.

\[ T_{BP} \leq A_{BP} f_{u,BP} \]  

(6-4)

where \( A_{BP} \) is the area of the base panel and \( f_{u,BP} \) is the tensile strength of the base panel material. To calculate the moment capacity \( M \) at the critical section of the hybrid steel deck, different theories can be employed.

**Simple-plastic theory**

The moment capacity can be calculated based on the simple-plastic rectangular stress block theory (also used in the partial shear connection theory for composite beams). It assumes that the top plate region, as well as the base panel region is stressed uniformly and that the resultant forces of these regions act at their individual centroids. The effect of the webs has been ignored for simplification which allows a two-force model to be considered. The horizontal equilibrium in the panel requires, that the resultant tensile force in base panel \( T_{BP} \) equals the resultant compressive force in top plate \( C_{TP} \) (see Figure 6-39).

![Figure 6-39. Cross-section and stress/strain distribution (simple-plastic theory)](image)

Therefore, the moment capacity of a panel exhibiting partial shear connection \( M \) can be calculated as follows

\[ M = T_{BP} (y_{TP} + y_{BP}) = n F_{TOX} (y_{TP} + y_{BP}) \]  

(6-5)

In contrast, the moment capacity of a panel exhibiting complete shear connection \( M_0 \) is not governed by the strength of the longitudinal shear connection but by the tensile strength of the base panel. At ultimate load, the panel will fail due to fracture (providing any secondary failure modes are eliminated) as seen in Chapter 6.4.2 and the moment capacity can be calculated as follows

\[ M_0 = A_{BP} f_{u,BP} (y_{TP} + y_{BP}) \]  

(6-6)

In Figure 6-40 and Figure 6-41, the normalized moment capacity \( M / M_0 \) is plotted against the nominal TOX spacing for specimens with a base panel thickness of 1.00 mm and 0.75 mm, respectively. The graphs include curves based on the partial shear connection strength theory, using simple-plastic theory, as well as the moment capacities \( M \) measured in each of the tests. The theoretical moment capacity of a panel exhibiting complete shear connection \( M_0 \) was calculated with the geometry of a standardized panel and the ultimate tensile strength was obtained from tensile coupon tests of the same material as used in the tests.
Chapter 6. Base panel shear tests

Figure 6-40. Comparison of test results with simple-plastic theory ($t_{BP}=1.00$ mm)

Figure 6-41. Comparison of test results with simple-plastic theory ($t_{BP}=0.75$ mm)

It can be seen that the partial shear connection strength theory predicts the trend well but is overly conservative when using simple-plastic theory. This can be explained by the fact that the rectangular stress block theory assumes the entire base panel to plastify when fracture occurs. This however, could not be observed in the tests where fracture was initiated at a single TOX location on one side only and slowly proceeded through the base panel to the other side (see Chapter 6.4.2). This assumption overestimates the resultant tensile force $T_{BP}$ at fracture which consequently means that a closer critical connector spacing $s_c$ is then required to achieve fracture. The design curve is not linear as the TOX spacing $s$ is an inverse function of the number of TOX ($s = L/n$ where $n$ TOX are spread over a length $L$). The calculated critical TOX spacing $s_c$ to achieve complete shear connection is
62 mm for material combination 10070 (57 mm for 10090) and 65 mm for combination 07570 and can be used to obtain the degree of shear connection $\beta$ for individual connector spacings ($\beta = s/l_s$).

**Linear-elastic theory**

As discussed above, plastic behaviour seems not to be appropriate for the components of the hybrid steel deck where the materials remain essentially in the elastic range over most of the load range and failure is by shear of the connection. Therefore, a conventional linear-elastic approach is considered to be more suited for these cold-formed panels although it does not account for longitudinal slip, strain hardening or any other non-linear effect. From elastic bending theory, the stress $f$ at the distance $y$ from the neutral axis is given by

$$f = \frac{M y}{I_p}$$

(6-7)

where $I_p$ is the second moment of area of the panel and $M$ the moment capacity.

At ultimate load, it can be assumed that only the outer fibre of the base panel will reach ultimate tensile strength $f_u$ and fail due to fracture. The stress $f_{u, BP}$ at the centroid of the base panel at fracture is therefore reduced to

$$f_{u, BP} = f_u \frac{y_{BP}}{y_S}$$

(6-8)

where $y_{BP}$ is the distance between the neutral axis and the centroid of the base panel, and $y_S$ is the distance between the neutral axis and the outer fibre of the base panel. In accordance with the partial shear connection strength theory, the resultant tensile force in the base panel $T_{BP}$ is the sum of the connector forces but cannot exceed the tensile capacity of the base panel. Hence, the stress at the centroid of the base panel can be calculated as follows

$$f_{BP} = \frac{T_{BP}}{A_{BP}} = \frac{nF_{TOX}}{A_{BP}} \leq f_{u, BP}$$

(6-9)

The moment capacity of a panel exhibiting partial shear connection $M$ can be expressed as

$$M = \frac{nF_{TOX}}{A_{BP}} \frac{I_p}{y_{BP}}$$

(6-10)

whereas the moment capacity of a panel exhibiting complete shear connection $M_0$ is given by

$$M_0 = f_u \frac{y_{BP}}{y_S} \frac{I_p}{y_{BP}} = f_{u, BP} \frac{I_p}{y_S}$$

(6-11)

**Linear-elastic theory neglecting effect of web**

A simple approach is to neglect the effect of the corrugated webs and to consider only horizontal force equilibrium of the top plate and base panel, as illustrated in Figure 6-42.
Chapter 6. Base panel shear tests

Figure 6-42. Cross-section and stress/strain distribution (lin.-el. Theory/no web)

In Figure 6-43 and Figure 6-44 the normalized moment capacity $M / M_0$ is plotted against the nominal TOX spacing for specimens with a base panel thickness of 1.00 mm and 0.75 mm, respectively.

Figure 6-43. Comparison of test results with lin.-el. Theory/no web ($t_{BP}=1.00$ mm)

Figure 6-44. Comparison of test results with lin.-el. Theory/no web ($t_{BP}=0.75$ mm)
The graphs include curves based on the partial shear connection strength theory, using linear-elastic theory and neglecting the effect of the web, as well as the moment capacities $M$ measured in each of the tests. The moment capacity of a panel exhibiting complete shear connection $M_0$ was calculated with the geometry of a standardized panel (ignoring the web) and the ultimate tensile strength was obtained from tensile coupon tests of the same material as used in the tests.

The critical TOX spacing $s_c$ to achieve complete shear connection is then 68 mm for material combination 10070 (64 mm for 10090) and 73 mm for combination 07570. These values are higher than the values obtained assuming simple-plastic behaviour as a result of the reduced tensile force $T_{BP}$ in the base panel to cause fracture. However, while the trend of the partial shear connection strength theory follows the test results, it can be seen that this approach is still very conservative and generally underestimates the moment capacity of the overall panel, suggesting that the web perhaps has an effect on the moment capacity. An increased value of $M_0$ from including the web would result in a lowering of the $M/M_0$ values.

**Linear-elastic theory considering a fully effective web**

This approach considers a fully effective web and therefore takes account of any moment arising from curvature of the individual plates. Since the linear-elastic theory ignores the development of longitudinal slip, it assumes the same strain levels in the web and in the base panel at their interface (see Figure 6-45).

**Figure 6-45. Cross-section and stress/strain distribution (lin.-el. Theory/full web)**

In Figure 6-46 and Figure 6-47, the normalized moment capacity $M/M_0$ is plotted against the nominal TOX spacing for specimens with a base panel thickness of 1.00 mm and 0.75 mm, respectively. The graphs include curves based on the partial shear connection strength theory using linear-elastic theory and considering fully effective webs of the panel as well as the moment capacities $M$ measured in each of the tests. The moment capacity of a panel exhibiting complete shear connection $M_0$ was calculated with the geometry of a standardized panel (including the web) and the ultimate tensile strength was obtained from tensile coupon tests of the same material as used in the tests.
The critical TOX spacing $s_c$ to achieve complete shear connection is 69 mm for material combination 10070 (64 mm for 10090) and 73 mm for combination 07570. The approach takes into account that the normalized moment capacity of the material combination with 0.90 mm web is less than with 0.70 mm web due to the lower shear strength capacity of the TOX connectors. It can be seen that the linear-elastic theory considering a fully effective web fits the test data very well, although slightly overestimating the moment capacity of the overall panel at a few points. This might be explained by the reduced in-plane bending stresses due to the corrugation of the webs compared to the stresses for the assumed solid webs. Another factor could be that the web was manufactured from a lower steel grade (G350) than the base panel (G550). At the levels of strain required to fracture the base panel, the lower parts of the web would then have yielded already although an elastic stress
distribution was assumed in the web to calculate $M_0$. Therefore, the value for $M_0$ used in the elastic theory considering fully effective webs may have been slightly overestimated. Using a lower value of $M_0$ would increase the values of $M/M_0$ in the graphs (Figure 6-46 and Figure 6-47), noting that removal of the entire web led to a more conservative result (Figure 6-43 and Figure 6-44). Irrespective, the results show that the web does have a significant influence on the behaviour.

6.8 CONCLUSION

A programme of laboratory tests has been carried out on three different material combinations to investigate the effect of connector spacing on the behaviour of the hybrid steel deck. The series included two different panel versions, viz. a configuration with anchored ends and with free ends. Various modifications were made to the panels to suppress unwanted failure modes without changing the behaviour of the shear connectors in the base panel. Fracture of the base panel indicated that a state of complete shear connection was achieved whereas longitudinal shear failure of the TOX connectors occurred for panels with partial shear connection. Generally, higher loads could be observed with thicker base panel material whereas an increase in the web thickness slightly reduced the ultimate load. A distinct relation between ultimate load and connector spacing was also noted. Vertical deflection, strain and longitudinal slip were measured for all specimens during testing. Measurements of longitudinal slip showed a fundamentally different distribution along the length of the panel compared to the conventional Newmark theory used for composite steel-concrete beams. A simple computational model, based on the Vierendeel truss analogue, was developed to model the slip behaviour of the panels and could be also used to predict the deformation performance of the hybrid steel deck which is mainly influenced by the development of longitudinal slip. The investigation on a faulty TOX connector in a critical region indicated that maintaining a regular connector spacing is essential to the proper performance of the hybrid steel deck. Finally, the ultimate moment capacity of the tested panels could be well predicted by using a partial shear connection model in conjunction with elastic theory, noting that neglecting the effect of the web would lead to more conservative results.
CHAPTER 7
MEASUREMENT OF TOP PLATE DEFORMATIONS USING CLOSE-RANGE PHOTOGRAMMETRY

7.1 GENERAL

Preliminary testing of the hybrid steel deck has shown that major out-of-plane distortions can occur when the top plate fails in compression. Several alternatives have been considered to measure these distortions, but most of them were not practical when considering the boundary conditions implied by the test method. Failure of the top plate typically involves a sudden overall buckle which could potentially damage any measuring device mounted above the top plate, and therefore precluded any measuring method with surface contact such as the commonly used linear potentiometers (LP’s).

An early attempt to overcome this problem was the development of the measuring rig shown in Figure 7-1. The rig allowed measurement of specific points on the top plate by having the measuring device mounted away from the critical area. Metal cables were glued onto the top of the top plate which were then redirected to LP’s vertically attached to the rig. Steel cylinders were connected to the end of the cables and experienced the same displacement as the measuring points.

![Figure 7-1. Measuring rig (left: side view, right: top view)](image)
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The measuring rig was based on the principle of gravity, with the steel cylinders designed slightly heavier than the spring resistance of the LP’s. Although this measuring method gave reasonable results, it was limited to a relatively small number of measuring points and was later disregarded in the search for a more sophisticated method.

To increase the number of measuring points, it is common practice to attach a set of measuring devices to a rigid frame and move it across the surface of interest, often by using a step-motor. This method is commonly used to determine initial imperfections where measuring time plays a minor role (Pircher and Wheeler, 2003), but seemed to be not suitable in this application where measurements had to be taken within a short time frame to avoid measuring errors. Another limiting factor for the selection of an appropriate measuring technique was the high curvature of the specimen during flexural testing which made the use of laser devices impossible. The advantage of this technique is that the device has no physical contact with the object to be measured but, on the other hand, requires a reasonable flat and shiny surface in order to reflect the laser beam back to its origin. This is all the more critical the further the laser device is located from the surface. Further research into optical measurement methods resulted in a technique called close-range photogrammetry.

The method involves photographing an object from several viewpoints without the change in the object geometry in the intervening period. In many cases, this can be done with a single camera moving around the object and the self-calibration of the camera is part of the standard measurement network. However, this was not practical for the present series of flexural tests since the object could not be considered to be static during the various loading epochs, and the time required to reposition the camera would have interrupted the loading procedure significantly. Consequently, a multi-station network with fixed camera positions on each side of the object was favoured and a self-calibration of each individual camera before testing was necessary.

7.2 BACKGROUND

"Photogrammetry is the science, and art, of determining the size and shape of objects as a consequence of analysing images recorded on film or electronic media. The word science is important, as it implies the laws of mathematics, physics and chemistry and a knowledge of their practical application. The ‘art’ in photogrammetry must not be overlooked. Good results can only be produced from suitable images, so photography and videography are skills which must not be dismissed lightly.” (Atkinson, 2001). Photogrammetry is a well established method of determining the three-dimensional geometry of objects (Fraser, 1992; Fraser and Brown, 1986) and the basic principles of photogrammetry are well documented elsewhere (ASP, 1980; Atkinson, 2001; Mikhail et al., 2001). It is probably best known for its use in aerial applications (e.g. mapping), but it is also used in many other fields such as Architecture, Engineering, Archaeology, Medicine, Forensics, Motion Caption and, of course, industrial measurement applications. Examples of structural engineering applications are deflection measurement of a bi-steel slab (Woodhouse et al., 1999), monitoring thermal deformation of steel beams (Fraser and Riedel, 2000), measurement of cracks in concrete structures (Dare et al., 2002) and measurement of geometric imperfections as well as buckled waveforms of cold-formed steel decking (Bernard and Coleman, 1993).
In the present study, digital close-range photogrammetry was employed to determine the three-dimensional spatial coordinates of each measuring point on the top plate during flexural testing. The term close-range photogrammetry is used to describe the technique when the size of the object to be measured is less than about 100 m and the camera is positioned close to it. Using digital camera technology has many advantages over traditional film-based cameras, in particular when photogrammetric software is used to process the data, but digital cameras with adequate resolution were, for many years, very expensive and reserved mainly to specialist users. Over recent years, digital camera technology became cheaper, mainly due to exploitation of the mass consumer market, and digital cameras in the range of 3 to 7 megapixel are now readily affordable making digital close-range photogrammetry more attractive to laypersons and researchers. The feasibility of low-cost digital cameras for close-range measuring systems has been previously examined (Acic, 2004; Chandler et al., 2005; Clarke and Robson, 1993; Fraser, 1997b; Fraser and Shortis, 1995). Based on these findings, close-range photogrammetric equipment and a software system for off-line digital close-range photogrammetric image measurement, orientation/triangulation and sensor calibration was purchased.

7.3 EQUIPMENT AND TEST SET-UP

The photometric measurement process for multi-station imaging networks involves traditionally the following phases: network design, image acquisition, image mensuration, interior orientation, and exterior orientation (Fraser, 1998). Network design plays an important role in the accuracy of any optical triangulation system, since it is a function of, firstly, the angular measurement resolution and, secondly, the geometry of the intersecting rays at each target point (Atkinson, 2001). With fixed camera positions, a minimum of four cameras was necessary to achieve a network of strong convergency. Since only the upper surface of the top plate had to be surveyed, the object was essentially two dimensional, and the generic network for measuring near planar objects could be used (Atkinson, 2001). The four-station configuration is defined by 90 degree intersection angles, and approximately 60 degree elevation angles. Four digital SLR cameras (Nikon D70s) with a high resolution CCD sensor have been purchased. Each sensor is 23.7 x 15.6 mm and contains 3008 x 2000 pixels, resulting in square pixels of 7.9 μm length. Square pixels are advantageous for metric applications because they eliminate distortion of the image. Since the photosensitive area of the chip is roughly one third smaller compared to a standard 24 x 36 mm film, special lenses are required for digital SLR cameras. The knowledge of the exact principal distance, which is the distance between the lens and the sensor, is of fundamental importance in photogrammetry and conventional zoom lenses are, in this regard, not optimal since the focal length and consequently the principal distance changes with the zoom setting (Mikhail et al., 2001). Therefore fixed focal length lenses (Nikon AF Nikkor 35 mm, f/2D) were chosen. The lenses have an approximate 35 mm focal length, which results in a viewing angle of about 62 degrees, and was considered to be sufficient to cover the measurement area. The measurement area was defined by the width of the top plate and should allow at least three buckling wavelengths to form. A distance of 2 m between the camera and the object seemed to be appropriate, yielding in an image scale of 1:60 (= 2 m / 35 mm). The following formula gives an
approximation of the object measurement accuracy attainable in a convergent, multi-station close-range photogrammetric network (Fraser, 1992):

\[ \bar{\sigma}_c = \frac{q}{\sqrt{k}} S \sigma \]  

(7-1)

where \( \bar{\sigma}_c \) is the root-mean-square (RMS) value of the XYZ co-ordinate standard errors; \( S \) is the image scale; \( q \) is an empirical design factor expressing the strength of the basic camera station configuration; \( \sigma \) the image co-ordinate standard error; and \( k \) the number of exposures recorded at or near each station in the network. Theoretically the measurement accuracy can be doubled if four exposures at each station are taken. However, this is not possible when the camera position, as in this test series, is completely fixed at the station since any minor systematic error that might be ‘averaged’ by small movements of the camera will likely be the same from image to image, and only a single exposure at each station \((k = 1)\) is usable. For strong network geometries, a design factor value of 0.7 can be assumed, and an image mensuration standard error equal to 10% of the sensor pixel size is relatively conservative. Applying these values to equation (7-1), the expected object space accuracy is then 0.033 mm.

Suitable targets had to be placed onto the top plate to ensure proper identification of the measurement points from all camera stations. Targets of circular shape are most suitable since the object circle is projected onto the image plane as an ellipse, if the object plane and the image plane are not parallel to each other, and allow automatic determination of its centroid. Depending on the method used, the precision of digital target location can be in the order of 0.02 pixel or better (Trinder et al., 1995). To guarantee reliable image processing and to limit the angular measurement error, the target diameter becomes of particular interest and a minimum/maximum criterion exists (Ahn et al., 1999). In practice, the mean diameter of the targets should be larger than 5 pixels, but not more than 25 pixels. To ensure a good contrast between the object surface and the circular target, retro-reflective targets are favourable. This can even be enhanced by slightly underexposing the photography and using less flash power so that the reflective targets appear bright against a very dark background. With an expected amount of 400 targets per specimen and an oblong measuring area, the use of pre-fabricated adhesive tapes with reflective targets at a nominated spacing was advantageous. The tapes were purchased from HUBBS MACHINE & MANUFACTURING in the U.S., and a target diameter of 0.25 inch (about 6.35 mm) was chosen. With an image scale of 1:60, the average target size in the image will then be about 0.1 mm, and corresponds roughly to 12 pixels. A nominal spacing of 1.00 inch (about 25.4 mm) was considered to be appropriate to even detect buckles with short buckling half-wavelengths of around 75 mm using only three measuring points. Typically, three lines of targets were applied onto the plate sub-element between the longitudinal stiffeners (middle part), and two lines onto each sub-element with edge-stiffener (north or south part). For better identification, each line of targets points was given a name, as shown in Figure 7-2.
With a long and narrow measuring area, the object had to be surrounded with non-moving targets to ‘fill up’ the image from each camera station and to get a good 2D spread of targets. These stable points are necessary to add geometric strength during the triangulation/bundle adjustment process and essential to measure deformation of a moving object by providing an absolute reference frame for all loading epochs. Based on the reference co-ordinate system displayed on the left in Figure 7-3, the absolute position of each stable point can be established which will then be used to transform the relative co-ordinates of the moving points into this reference system. In doing so, each set of target locations refers to the same co-ordinate system and makes a direct comparison between the loading epochs possible. The incremental deflection of a specific measurement point is then the difference in vertical direction between two loading epochs. For this reason, a full set of images with the reference co-ordinate system placed on top of the top plate is taken before each flexural test. Three adjustable feet allow its horizontal alignment, using a conventional spirit level. Three retro-reflective targets form the XY-plane of the reference co-ordinate system, with the origin O located over the centre of the plate at midspan. An explicit Z-direction is not necessary and will be automatically computed as the normal to the XY-plane when the images are processed. Presuming a horizontal XY-plane, the Z-axis will coincide with the vertical. The introduction of an exterior orientation (EO) device allows automatic resection and subsequent triangulation/bundle adjustment during image mensuration (Fraser, 1998). The device in this test series consists of five retro-reflective targets, surrounded by an unbroken line, to make it recognizable by the computer software. The target in the centre is slightly elevated to add a third dimension, as seen on the right in Figure 7-3.

The EO device can also be used to introduce a scale in the object space since photogrammetry captures only the shape of the object by establishing relative relationships between individual targets. This however requires the co-ordinates of each target on the EO device beforehand. With the aim of
measuring absolute deflections of the tested specimens, it was necessary to increase the accuracy of the scale in the object space due to the relatively small dimensions of the EO device. For this reason, a more accurate reference scale made out of steel was introduced, which had a calibrated point-to-point distance of 950 mm and a standard deviation of 0.15. The final arrangement of the stable points, the EO device and the reference scale alongside the failed specimen is displayed on the left of Figure 7-4.

Tests were carried out in the Structural Engineering workshop of the University of Western Sydney, Australia. For safety reasons, overhead lights in the workshop had to be kept on all day, creating unwanted reflections and hot-spots on the metallic surface of the top plate which in return had a negative influence on the quality of the images for the photogrammetry. For this reason, it was decided to build a ‘tent’ structure around the test apparatus using thick black builder’s foil, as pictured on the right in Figure 7-4, which could also be used to mount the four digital SLR cameras. The structure allowed easy access to the specimen and created a dark environment, independent of sunlight and the day-to-day activities in the workshop. One camera was positioned at each corner of the measuring area approximately 1.4 m above the target plane, resulting in a distance of about 2 m from the camera to the centre of the measuring area and intersecting angles of approximately 90 degrees between the cameras. From these camera positions, the majority of the retro-reflective targets in the measuring area were visible and in focus during all stages of the test. The reference scale and the EO device were positioned near the centre of the measuring area to ensure visibility throughout the whole test. The displaying imaging rays from all four camera stations for selected targets of a typical specimen can be seen in Figure 7-5.

![Figure 7-4. Final arrangement of auxiliary targets (left) and ‘tent’ structure (right)](image)

### 7.4 CAMERA CALIBRATION

The main objective for a camera calibration is the determination of the optical and geometric parameters of a camera-lens system, with the principal point as one of the most fundamental parameters (Clarke et al., 1998). With every camera-lens system being different, it is important for photogrammetric purposes to know its internal geometric configuration, also known as interior orientation. This orientation can be mathematically formulated by a set of parameters which describes the physical reality when a bundle of light rays coming from an object passes through the lens to the image plane of the imaging device.
Since four different camera-lens systems at a fixed camera position were used in this test series, an ‘on-the-job’ calibration method was not feasible and the interior orientation of each system had to be established before the actual test series. This was accomplished by using the self-calibration method in which the parameters of the camera calibration and the actual object point co-ordinates are determined simultaneously. A more detailed description of the analytical self-calibration process applied to digital cameras is given elsewhere (Fraser, 1997a). For each camera-lens system, pictures from eight convergent camera stations were taken of an area with discrete target points covering most of the image format, as shown on the left in Figure 7-6. To improve the calibration process, two exposures were made at each station with one being rotated by 90 degrees, as illustrated in the displaying imaging rays for selected targets on the right in Figure 7-6.
Since the focal length and consequently the principal distance changes with different focus settings, the camera calibration is only valid for observations with the same focus. For this reason, the calibration process was performed with a focal setting suitable for the test series, and kept the same for all subsequent tests. With minor variations in distance from the camera to the target points, increasing the depth of field of the camera was advantageous to ensure all moving and stable targets were in sufficiently good focus.

Research has shown that a 10-parameter model for the interior orientation can result in object space triangulation accuracies of well beyond 1:100,000 (Frasier, 1997a). The model consists of the following parameters to define the camera characteristics: the principal distance \( c \), the principal point offsets \( x_p \) and \( y_p \), the coefficients of radial lens distortion \( K_1 \), \( K_2 \) and \( K_3 \), the decentring distortion parameters \( P_1 \) and \( P_2 \), and finally the empirical correction terms for sensors with digital output \( b_1 \) and \( b_2 \). For the current camera self-calibration, such a 10-parameter model was employed, starting with a nominal focal length of 35 mm. With the knowledge of having square pixels on the sensor of the Nikon D70s, the parameters \( b_1 \) and \( b_2 \) were a priori set to zero. The calibration was accepted once the root-mean-square value of the image co-ordinate residuals (RMS) was better than 1/20\(^{th}\) pixel, i.e. 0.39 \( \mu \)m. The results of each camera self-calibration are listed in Table 7-1.

<table>
<thead>
<tr>
<th>Camera</th>
<th>( c ) (mm)</th>
<th>( x_p ) (mm)</th>
<th>( y_p ) (mm)</th>
<th>( K_1 )</th>
<th>( K_2 )</th>
<th>( K_3 )</th>
<th>( P_1 )</th>
<th>( P_2 )</th>
<th>RMS (( \mu )m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>36.333</td>
<td>0.179</td>
<td>0.158</td>
<td>6.94e-5</td>
<td>-7.47e-8</td>
<td>9.63e-11</td>
<td>-5.97e-6</td>
<td>-1.04e-5</td>
<td>0.35</td>
</tr>
<tr>
<td>2</td>
<td>36.418</td>
<td>0.016</td>
<td>0.167</td>
<td>7.25e-5</td>
<td>-9.94e-8</td>
<td>1.92e-10</td>
<td>5.49e-6</td>
<td>-1.90e-5</td>
<td>0.35</td>
</tr>
<tr>
<td>3</td>
<td>36.357</td>
<td>0.012</td>
<td>0.098</td>
<td>7.03e-5</td>
<td>-6.75e-8</td>
<td>4.48e-11</td>
<td>1.18e-6</td>
<td>-1.29e-5</td>
<td>0.38</td>
</tr>
<tr>
<td>4</td>
<td>36.372</td>
<td>0.090</td>
<td>0.029</td>
<td>7.07e-5</td>
<td>-7.68e-8</td>
<td>7.92e-11</td>
<td>-2.32e-5</td>
<td>-6.61e-6</td>
<td>0.35</td>
</tr>
</tbody>
</table>

### 7.5 IMAGE MENSURATION

The commercial available photogrammetric software package AUSTRALIS (AUSTRALIS, 2004) was employed to determine the three-dimensional spatial coordinates of the top plate during flexural testing in this test series. The software has been developed primarily as both an educational tool and as a platform to support applied research. The user-friendly software runs on a conventional personal computer (PC) system and allows even non-specialist users of close-range photogrammetry to perform automated operations such as single- or multi-sensor, multi-station orientation/triangulation, calibration and point-based 3D object measurement (Fraser and Edmundson, 2000). In using high-contrast targets, i.e. retro-reflective targets, it is possible to execute an automated image scan and centroid measurement of all candidate targets, with an image mensuration precision of 0.02–0.04 pixel (Fraser and Edmundson, 2000), as opposed to a maximum accuracy of 0.3 pixel, when the feature points are selected manually. Although the ‘AutoScan’ function provides the possibility of an automatic target point labelling, manual labelling was inevitable to ensure a consistent name pattern and made a direct comparison of distinct targets between various loading epochs possible. With approximately 400 target points per specimen, which were mostly regularly spaced along straight lines, this process could be semi-automated by using the ‘LineFollowing’ function. Figure 7-7 shows a test specimen close to failure observed from all four camera stations, with individual point labels assigned to each retro-reflective target.
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All images were taken upside down, due to the way the digital SLR cameras were mounted to the ‘tent’ structure, in order to allow easy access to the cameras’s function keys during testing.

Spatial intersection (or bundle adjustment) of the image rays from the four camera stations determined the position of all target points in the object space. An image ray is the most elementary geometric unit in photogrammetry, and connects an object space point with the projection of the point on the image via the perspective centre of the image. A single image can be characterized by a bundle of such rays which converges at the perspective centre with unknown position and orientation in space. The position and orientation of each bundle in respect to the object space coordinate system (exterior orientation) can then be established by a bundle block adjustment which is the most accurate and flexible method of triangulation (Mikhail et al., 2001). The bundle adjustment in this test series was performed without additional control points using a free network solution. To make a direct comparison between individual loading epochs possible, the co-ordinates had to be transformed into an absolute reference frame using the previous described reference co-ordinate system. The initial origin of the reference co-ordinate system was set at the centre of the middle section at midspan at a known height (approximately 35 mm) above the top plate of the panel. In processing the data, this height was removed from all data points to set the origin at the top of the middle section at midspan. The vertical displacement $\delta$ of a specific measurement point at the loading epoch $i$ could then be determined by subtracting the vertical point position $z$ at the initial loading epoch $o$, i.e. when no load was applied, from the vertical point positions $z$ at the loading epoch $i$.

$$\delta = z_i - z_o$$ (7-2)
Displaying the discrepancy vectors of all targets between the loading epochs \( o \) and \( i \), allows a graphical examination of the correct image mensuration. In Figure 7-8, all moving targets show vertical displacements only, represented by the vertical lines, whereas the stable points did not experience any movement at all.

![Figure 7-8. Discrepancy vectors between loading epochs \( o \) and \( i \)](image)

The estimated accuracy for the XYZ co-ordinates at a specific loading epoch is expressed in the root-mean-square (RMS) value of the standard error. With images taken and processed independently for each loading epoch, these standard errors varied slightly and are summarized in Table 7-2 for the Z-direction, i.e. vertical direction.

**Table 7-2. RMS value of object point coordinate standard error in Z-direction**

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Spacing (mm)</th>
<th>RMS (Initial) (mm)</th>
<th>RMS (Final) (mm)</th>
<th>RMS (Post) (mm)</th>
<th>( \sigma_{\text{m. Final}} ) (mm)</th>
<th>( \sigma_{\text{m. Post}} ) (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>90250B70-100</td>
<td>100</td>
<td>0.0246</td>
<td>0.0212</td>
<td>0.0262</td>
<td>0.0325</td>
<td>0.0359</td>
</tr>
<tr>
<td>90250B70-120T</td>
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<td>0.0302</td>
<td>0.0358</td>
<td>0.0448</td>
<td>0.0488</td>
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<td>0.0334</td>
</tr>
<tr>
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<td>0.0388</td>
</tr>
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</tr>
<tr>
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<td>0.0292</td>
<td>0.0349</td>
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<td>0.0351</td>
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<td>0.0281</td>
<td>0.0301</td>
<td>0.0391</td>
<td>0.0406</td>
</tr>
</tbody>
</table>

For the initial loading epoch, accuracies between 0.019 and 0.033 mm could be achieved with similar values for the final loading epoch (i.e. the last recorded epoch prior failure) and for the post buckling epoch. The obtained errors fit well with the previous calculated expected object space
accuracy of 0.033 mm (refer to Chapter 7.3). However, since the vertical displacement $\delta$ is the difference of the $z$-values between two loading epochs, the overall measurement error $\sigma_m$ is the accumulation of the individual errors in each loading epoch. In this test series, the overall measurement error ranges from 0.037 mm to 0.045 mm, and was obtained through following equation.

$$\sigma_{m,i} = \sqrt{\sigma_0^2 + \sigma_i^2}$$

(7-3)

7.6 DATA PROCESSING

Photogrammetry is a very powerful tool for establishing the spatial coordinates of pre-defined targets to a high level of accuracy. These coordinates can then be used in different ways to obtain information about the observed object. In the case of the hybrid steel deck, the data collected for the top plate was displayed in four different ways (refer to Chapter 8.4.2 and Appendix F). Firstly, all the data points can be plotted together giving an overall picture of the global deformation of the top plate, as displayed in Figure 7-9. The 3D-contour plots (with crosses indicating the actual measurement points) were created by using the commercial contouring and mapping software SURFER (SURFER 7, 1999). Secondly, individual longitudinal lines can be extracted and plotted for each line of targets, as shown in Figure 7-10. Thirdly, the data can also be used to obtain points on any cross-section desired (Figure 7-11). In addition, a curve of best fit (in this case a parabola) can be fitted to longitudinal lines of data to obtain the global curvature of the data. Subtracting this curvature from the data points enables the local variation of deformation to be determined, as shown in Figure 7-12. Hence, data can be displayed in this manner for data obtained from sets of photographs taken before loading (initial imperfections) or during loading (loading induced deformation).

Initial imperfections

The global deformation relative to the ends for specimen 90160B70-100 is in the order of about 1 mm (see Figure 7-10). This curvature is mainly due to the self-weight of the panel as the initial measurement was taken with the panel already placed into the testing apparatus where it was supported only at its ends. The local imperfections from the overall curved state are relatively small, as shown in Figure 7-12 for the two worst longitudinal lines on the middle and south sub-element of the top plate. For virtually all the points, deviations are less than 10 % of the plate thickness $t_b$. The cross-section shown in Figure 7-11 indicates that the top plate of the panels were not entirely flat as intended which is a direct result of the roll-forming process during manufacturing. In particular the thinnest top plate experienced distortion of the plate sub-elements on the south and north side, and therefore would have the largest initial imperfections. Note that on the graph, the scale in the vertical direction is amplified to highlight the imperfections and the deviations were less than 2 mm over a height of 90 mm for the panel.
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Figure 7-9. 3D-contour plot of specimen 90160B70-100 before loading

Figure 7-10. Longitudinal lines of specimen 90160B70-100 before loading
Figure 7-11. Cross-section of specimen 90160B70-100 before loading

Final buckling shape

Figure 7-13 to Figure 7-16 show the deformation under load for the last data set obtained just prior to buckling of the top plate of specimen 140185L70-100. Global deformations in Figure 7-13 show the formation of a buckle on the south side of the panel. This can be more clearly seen in Figure 7-14 which shows the deformations of the plate sub-element on the south side with the maximum deformation at the edge. It includes the global curvature of the worst longitudinal line, obtained from a curve of best fit to the data points which gives a clearer picture of the local deformations due to buckling. The local deviations are then obtained by subtracting the global curvature from the data points of individual longitudinal lines. In Figure 7-15, this was done for the worst longitudinal line observed in the final buckling shape and compared with the initial imperfections of the same line. This can be used to indicate whether imperfections were significant in the formation of the buckles as well as to determine the wave length and/or amplitude of the local buckles. Figure 7-16 shows the effect of buckling on the distortion of the cross-section at three locations along the length of the panel. In utilising photographs taken after the failure of the top plate, the post-buckled shape can be compared with the local deformations of the worst longitudinal line (see Figure 7-17) to verify if the overall plate buckle occurred at the same position as the most pronounced local buckle. Shown in Figure 7-18 are the global deformations of the base panel obtained from LP’s and through photogrammetry. The results show good agreement giving confidence in the photogrammetry technique. Due to the high accuracy of the photogrammetry, any measurement differences are more likely because of the non-vertically of the placement of the LP’s, the reset of LP’s when they reached their limit, and the fixed position of LP’s which do not follow any lateral or longitudinal movement of the panel.
Chapter 7. Measurement of top plate deformations using close-range photogrammetry

Figure 7-13. 3D-contour plot of specimen 140185L70-100 just prior failure

\[ y = 0.0000110694x^2 - 0.0005215077x - 66.1007434038 \]

Figure 7-14. Longitudinal lines of specimen 140185L70-100 just prior failure

Figure 7-15. Final buckling shape and initial imperfections of specimen 140185L70-100
Chapter 7. Measurement of top plate deformations using close-range photogrammetry

Figure 7-16. Cross-section of specimen 140185L70-100 just prior failure

Figure 7-17. Post-buckled shape vs. worst longitudinal line of specimen 140185L70-100

Figure 7-18. Comparison of base panel deflection measurements (specimen 140185L70-100)
7.7 CONCLUSIONS

The measurement of the out-of-plane distortions of the top plate played an important role in understanding the failure behaviour of the hybrid steel deck. Various measurement techniques were considered, but digital close-range photogrammetry has been shown to provide highly accurate and extensive data of initial imperfections and deformations, in particular shapes of local buckles, of the top plate during testing. The technique gives the opportunity to survey any number of targets at desired positions and has no physical contact with the object. The data collection itself is quick but data processing can be time consuming and care has to be taken in setting up the equipment. However, in using off-the-shelf digital cameras and a suitable photogrammetric software package, even a non-specialist user of the photogrammetric technique could achieve highly accurate measurements up to 0.02 mm in this application.
8.1 GENERAL

During preliminary testing of the hybrid steel deck (Chapter 4), various major failure modes possible during flexural bending have been identified with a buckling failure of the top plate as one of them. As the hybrid steel deck undergoes flexural bending, the base panel is in tension whereas the top plate is in compression and therefore subject to buckling. The hybrid steel deck consists of individual elements (including the top plate) which are discretely connected to each other. Since discrete connectors provide less restraint than continuous connection (such as continuous seam-welds), more local flexibility exists which can lead to local buckling of the compression elements if the spacing of the connectors is too large. An early investigation into the local buckling behaviour of intermittently welded structural members has shown that a very close spacing is needed to suppress local buckles altogether (Norris and Scalzi, 1954). However, using more connectors than strictly necessary can lead to a significant increase in production time and cost. Therefore, the manufacturer would ideally like to know the minimum number of connectors needed for a safe and economical design.

The Australian and American Cold-Formed Steel Standards (AISI-2001, 2003; AS/NZS 4600, 2005) give guidelines regarding the spacing of welds, bolts or rivets connecting a cover plate, sheet or non-integral stiffener in compression to another element. According to these guidelines, the spacing should be limited: (a) to that which is needed to develop the required shear strength; (b) to avoid column-like buckling behaviour of compression elements between connection; and (c) to prevent possible buckling of unstiffened elements between the centre of the connection lines and the free edge.

Although the standard does not closely specify the geometry of the cover plate, the guidelines are made for a flat sheet of thickness $t$ without longitudinal intermediate or edge stiffeners that acts monolithically with the cross-section. In particular, criterion (b) can therefore be very conservative for
longitudinal stiffened elements. For instance, to achieve a compressive stress of 300 MPa in a 2.5 mm thick top plate of a 90 mm high hybrid steel deck section, the standard recommends a connector spacing of 75 mm to prevent column-like buckling, based on the assumption of a flat cover sheet. However, using the appropriate radius of gyration for the stiffened plate in the Euler equation for column buckling (on which the second criteria is based (AISI-1968 Comm, 1976)), the connector spacing changes to 445 mm which is an increase of almost six times.

A review of available literature has shown that spacing of connections in compression flanges of built-up sections has not been studied much in the past and was mainly focused on flat cover plates without longitudinal intermediate stiffeners (Jones et al., 1997; Luttrell and Balaji, 1992; Tillman, 1992; Yener and White, 1984). While the study at the University of Missouri-Rolla (Jones et al., 1997) was initiated to determine if the above spacing criteria accurately predict the capacity of built-up sections with flat cover plate in compression (noting also that cover plates with edge stiffener were investigated), Yener proposed an alternative spacing criterion (Yener, 1984) where the spacing shall not exceed: (a) that which is needed to develop the required shear strength; (b) that required to prevent separation of the compressed cover plate; and (c) that required to prevent separation of unstiffened compression plate elements.

The reasoning behind Yener’s design proposal is that regardless of how closely the connections are spaced, the buckling of the cover sheet as a plate (rather than a column) may not be prevented which is primarily dependent on the rotational restraint provided by the connectors and on the width-to-thickness ratio of the flat cover sheet. As longitudinal intermediate stiffeners subdivide the cover plate into several sub-elements, the ultimate capacity may be significantly increased and the buckling behaviour of the cover plate is further complicated due to the existence of local as well as distortional buckling modes (Schafer and Peköz, 1998), hence Yener’s proposal is not considered appropriate.

Criterion (c) is based on the finding that the outstanding unstiffened flanges of the connected fluted sheet are also in compression and tend to buckle at smaller stresses than corresponding stiffened plates. The buckling half-wavelength of this unstiffened plate element is directly proportional to the elastic restraint provided by the supported edge. Buckling of the element can then be prevented by placing one extra connection within the buckling half-wavelength. During preliminary testing of the hybrid steel deck, it was observed that buckling of the unstiffened upper web flanges, as seen in Figure 8-1, already initiated at relatively low compressive forces.

![Figure 8-1. Buckling of unstiffened upper web flange during flexural bending](image)
These local buckles, however, seemed not to have an influence on the buckling capacity of the top plate and might be explained by the fact that the top plate is usually much thicker (about 2.5 to 4 times) and stiffer than the unstiffened web flange. It is also clear from the testing that the unstiffened upper web flanges, even in the buckled state, are able to transmit the longitudinal forces from the web to the top flange through the connectors up to the point of buckling of the top plate. Therefore, the criterion of separation of the unstiffened compression web plate elements is not considered significant for the hybrid steel deck.

In order to investigate the buckling behaviour of the top plate and the influence of connector spacing, a programme of laboratory tests has been carried out. The top plate deformations before and during flexural testing were obtained through a non-contact measurement technique called digital close-range photogrammetry. Simple design models, using the finite strip method and an elastic member buckling analysis, have been established to predict the observed buckling behaviour of the top plate.

8.2 TEST SET-UP

8.2.1 General

The primary requirement in this test series was to generate high compressive forces in the top plate of the hybrid steel deck to study the buckling behaviour of this longitudinally stiffened and discretely connected plate. A very effective method to generate compressive forces in the top flange of a beam is through flexural bending. Uniform compressive forces can be achieved by keeping the bending moment constant over a certain length. The simplest way to obtain a constant moment region is by using a two-point load in a single span, with the loading points far enough apart to allow several buckling waves to form in this region. Different ways of applying the load were considered, and it was soon realized that loading the top plate was disadvantageous, mainly since it would impede the top plate in developing buckling waves. Applying the load to the top plate would also require more web reinforcement at the loading points to avoid secondary failure modes. The test set-up with a two-point load was also beneficial in using close-range photogrammetry as a non-contact measurement method (as described in more detail in Chapter 7).

8.2.2 Test apparatus

All tests in this series have been conducted as single span tests with a distributed two-point load. To allow horizontal movement of the specimen during bending, roller bearings were used as supports, and a 200 mm wide and 10 mm thick plate was fitted over the roller supports to better distribute the reaction forces. Each loading point was supplied with two standard loading rigs consisting of a crossbeam, two loading fingers and a loading bracket, as described in Chapter 4.2. The distance between the two loading points was 2,050 mm, and each loading point was supplied with two standard loading rigs to avoid concentrated loading of the sheeting and consequently premature failure of the press-joints. The loading rigs were rigidly connected to a spreader beam at a distance of 150 mm which was attached to the loading beam via a pivot point. This arrangement provided 1,900 mm of
Chapter 8. Top plate buckling tests

constant moment region at midspan, allowing the top plate to form approximately four buckles with a buckling wavelength of approximately 450 mm. A shear span of 1,500 mm on each side was considered to be sufficient to transfer the longitudinal shear forces. Small metal packers approximately 25 mm long and 3 mm thick were used in between the press-joints to firstly provide an even area for the loading brackets and secondly not to directly load the connectors which could possibly damage them. The jack load was applied through a 1000 kN capacity MTS hydraulic actuator with a 250 mm stroke that allowed the use of pre-programmed stroke rates. Tests were carried out under position-control and the rates were usually in the range of 4 to 6 mm/min during the loading process and 6 to 10 mm/min during the unloading process. A schematic view of the loading arrangement can be seen in Figure 8-2 whereas Figure 8-3 shows the actual test apparatus.

![Figure 8-2. Loading frame with distributed two-point load – Side View](image)

For safety reasons, overhead lights in the workshop had to be kept on all day, creating unwanted reflections and hot-spots on the metallic surface of the top plate. This had a negative influence on the
quality of the images taken for the close-range photogrammetry, and it was decided to build a ‘tent’ structure around the test apparatus using thick black builder’s foil (see Figure 8-4). The structure allowed easy access to the specimen and created a dark environment, independent of sunlight and the day-to-day activities in the workshop.

8.2.3 Instrumentation and processing of test results

With expected loads below 50 kN, an external 100 kN load cell (Type TML-TCLM-10B) was used to measure the total applied jack load. The load cell was previously calibrated according to the NATA (National Association of Testing Authorities, Australia) specifications. Linear potentiometers (LP’s) of the type SAKAE-S18FLPA100R with a range of 100 mm were used to measure vertical deflection at three distinctive points across the base panel at midspan and on the north and south side near the loading points. All specimens had at least three strain gauges (120 Ω resistance gauges, produced by TOKYO SOKKI) attached to the outside surface of the top plate at midspan and on the north and south side. The strain gauges on the edge plate elements were centrally placed between TOX connectors to minimise the effect of potential stress concentrations around the connectors. Due to the closed cellular shape of the hybrid steel deck and the way the panels were manufactured, it was not possible to attach strain gauges on the inside surface of the top plate. For most specimens, additional strain gauges (120 Ω resistance gauges) were attached to the upper and lower part of the corrugated web as well as to the bottom of the base panel at mid-span. The data were used to obtain information about the strain distribution over the depth of the section during bending as well as to estimate the stresses achieved in the top plate. All test results, together with the exact strain gauge locations for all specimens are provided in Appendix H. For the close-range photogrammetry, retro-reflective targets with a diameter of 0.25 inch (about 6.35 mm) were glued onto the surface at a nominal spacing of 1.00 inch (about 25.4 mm). This spacing was considered to be sufficient to even detect buckles with short buckling half-wavelengths of around 75 mm using only three measuring points. Instrumentation of the test panels is illustrated in Figure 8-5 and details of the test set-up shown in Figure 8-6.
Deflection, load and strain gauge measurements were captured using a data acquisition system (DATA TAKER DT605, Series 3 for strain gauge measurement, and DATA TAKER DT800 for remaining measurements) and the software package DELOGGER PRO. Pictures of the retro-reflective targets were taken prior testing and at various loading epochs during the loading process of the specimens. Four digital SLR cameras (Nikon D70s) with fixed focal length lenses (Nikon AF Nikkor 35 mm, f/2D) were used to capture the images from different positions and mounted to the ‘tent’ structure (see Figure 8-6). The software package AUSTRALIS (AUSTRALIS, 2004) was used to process the photogrammetric information and made it possible to precisely determine the global position of each individual target. 3D-contour plots of the top plate deformation were created by using the commercial contouring and mapping software SURFER (SURFER 7, 1999).

### 8.3 TEST PROGRAM

#### 8.3.1 General

The test program was designed to cover the most important aspects regarding the top plate buckling failure mode, with the main focus on the spacing of the top plate connectors. The maximum connector spacing is a significant factor in terms of safety and economics of the panel, as increasing the spacing would lead to a faster manufacturing of each panel and consequently save production costs. This increase of connector spacing is limited by the local buckling behaviour of the top plate acting as a compression element, due to the reduced restraint given by the web. Various secondary
Chapter 8. Top plate buckling tests

Failure modes were considered and measures were taken to avoid them. A shear failure of the connectors in the base panel was prevented by having the TOX connectors in the shear span spaced very closely and using the thickest material available for the base panel, i.e. 1.00 mm, as it provides the strongest connectors in longitudinal shear. Shallow section heights, i.e. TD 90 or TD 110, were preferred in this test series since the stocky sections in combination with end-diaphragms would prevent a lateral-distortional sway mode failure. By using shallow and solid webs, i.e. no web penetrations, the possibility of a web buckling failure was more than unlikely. It was also believed that these shallow section heights are the more critical, in regards to top plate buckling, since they have the widest top plate and the widest sub-elements between the longitudinal stiffeners. In a first series, different top plate thicknesses made from BLACKFORM steel (the standard grade for the top plate in commercial applications) were under investigation. Two thicknesses (2.5 mm and 1.6 mm) were chosen to cover the upper and lower range used in commercial applications. BLACKFORM steel is a hot-rolled low carbon steel with no guaranteed minimal yield strength. However, the yield strength is typically in the range of 250 MPa to 380 MPa. For this reason, a second series was considered to be necessary to investigate different grades of the top plate material. It also included the influence of different web thicknesses (and hence restraint) in regards to the buckling behaviour of the top plate.

All panels in this test series had to be manufactured well before the actual testing to avoid conflicts with the commercial manufacturing line. Similar section geometries were considered to be advantageous to make a direct comparison possible, but stock available at the time of manufacturing was limited. Top plate material of different steel grades and similar thicknesses were only available for a 140 mm section height and therefore used in the second test series. The possibility of secondary failures, such as web buckling and lateral-distortional swaying, were more likely with higher webs but these were accounted for by reinforcing the section over the support points. For the first test series, a section height of 90 mm was chosen since it was the most common section height in actual applications and suitable web material was only available for this height.

During the course of testing, it was soon realized that it was not possible to distinguish if the final top plate buckling failure was caused by an initial pull-out failure of the TOX connectors or actually due to buckling or yielding of the top plate, although it was considered to be the latter. The TOX joints of the top plate between the loading points were therefore replaced with M5 bolts, simulating a TOX joint which would not fail in tension, thereby ensuring that failure would be in the plate panel. The strength of TOX connectors in tension has already been discussed in Chapter 3.1.5 and a method for determining separation forces in the connectors has been developed (Robinson and Naraine, 1988).

A distinct alpha-numeric code, determined from basic information, was given to each specimen. The initial two/three numbers characterize the nominal section height in millimetres, viz. 90 for a TD 90 section and 140 for a TD 140 section, followed by three numbers indicating the nominal thickness of the top plate (in millimetres times 100). The following letter represents the material grade of the top plate, where L stands for low-strength material (HA250), H for high-strength material (HA70T) and B for BLACKFORM material. The subsequent two numbers state the nominal thickness of the web (in millimetres times 100). The spacing of the top plate connectors at midspan is given in
millimetres and separated by a dash. If the test was carried out with the original TOX connectors (instead of having the bolt replacement), the letter T was added after the connector spacing.

Examples of the code are shown below.

<table>
<thead>
<tr>
<th>Section height</th>
<th>Top plate thickness</th>
<th>Grade</th>
<th>Web thickness</th>
<th>Connector spacing</th>
<th>TOX connectors</th>
</tr>
</thead>
<tbody>
<tr>
<td>140</td>
<td>185</td>
<td>L</td>
<td>90</td>
<td>200</td>
<td>T</td>
</tr>
</tbody>
</table>

8.3.2 Specimen details

The panels tested in this series were custom-made at PREMIER STEEL TECHNOLOGIES Pty. Ltd., NSW, Australia, using the TOX machine described in Chapter 2.4.2. The following tools were used to form the 8 mm TOX:

Table 8-1. TOX manufacturing tool sets

<table>
<thead>
<tr>
<th>Material punch side</th>
<th>Material die side</th>
<th>Punch No.</th>
<th>Die No.</th>
<th>Pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.50 mm (BLACKFORM)</td>
<td>0.70 mm (G350)</td>
<td>AS2100</td>
<td>BC8020</td>
<td>270 bar</td>
</tr>
<tr>
<td>1.60 mm (BLACKFORM)</td>
<td>0.70 mm (G350)</td>
<td>AS2100</td>
<td>BD8014</td>
<td>270 bar</td>
</tr>
<tr>
<td>1.90 mm (HA70T)</td>
<td>0.70 mm or 0.90 mm (G350)</td>
<td>AS2100</td>
<td>BC8020</td>
<td>270 bar</td>
</tr>
<tr>
<td>1.85 mm (HA250)</td>
<td>0.70 mm or 0.90 mm (G350)</td>
<td>AS2100</td>
<td>BC8020</td>
<td>270 bar</td>
</tr>
<tr>
<td>1.00 mm (G550)</td>
<td>0.70 mm or 0.90 mm (G350)</td>
<td>A60100</td>
<td>BB8012</td>
<td>200 bar</td>
</tr>
</tbody>
</table>

All sections had a 1.0 mm thick base panel (G550) and either a nominal height of 90 mm or 140 mm between the inside surface of the top plate and the base panel. The shape and nominal dimensions for the TD 90 sections are shown in Figure 8-7, and in Figure 8-8 for the TD 140 sections.

![Figure 8-7. Nominal dimensions of TD 90 sections](image-url)
The exact shape of the top plate was obtained by making casts of the outer surface of the plate using CIVILCAP material. CIVILCAP material is typically used to cap concrete compressive test cylinders and was adopted for this purpose. The material is delivered in granulates and melts when heated at a temperature of around 70°C. At this stage, the liquid was poured into metal moulds placed on top of the top plate, where it quickly set and became solid, as seen in Figure 8-9.

Figure 8-9. Casting of a top plate (left: metal mould; right: final cast)
Generally two casts (400 mm east and 400 mm west from midspan) were made for each specimen of this test series, as shown in Figure 8-10. Appendix G provides the traced outline of each cast including the relative position of the TOX connector.

![Casts of individual top plates (left: 2.50 mm; right: 1.90 mm)](image)

**Figure 8-10. Casts of individual top plates (left: 2.50 mm; right: 1.90 mm)**

During manufacturing, the following minor modifications were made to the panels:

- Thin-gauge, pressed open diaphragms were positioned at the intended support points (500/250 mm from the panel ends of the TD 90/TD 140 section) as well as at the intended loading points (1,800/1,550 mm from the panel ends of the TD 90/TD 140 section). Self-piercing rivets (HENROB) were used to attach the diaphragms to both the web and the base panel.

- Various TOX spacings were used at different regions of a specimen. In the region between the intended loading points, the spacing was kept constant and ranged from 70 mm to 140 mm, the latter regarded as the upper limit in an actual application. A much closer spacing was chosen for the 1.5 m shear span to transfer the longitudinal shear forces in the base panel without failure of the TOX joints at ultimate load. For this reason, 1.00 mm base panels were used for all test specimens as they provide the strongest connectors in shear. The following spacings were considered to be sufficient to achieve a top plate buckling failure: 60 mm for a section with a 2.5 mm top plate, 80 mm for a 1.60 mm top plate, and 55 mm for a 1.85/1.90 mm top plate. As is standard practice, four TOX joints at a spacing of 20 mm were located at the end of each panel.

### 8.3.3 Mechanical properties

Tensile testing of coupons was carried out to determine the mechanical properties of each material used in the test series, following the guidelines of the Australian Standard (AS 1391, 1991). Material for the base panel and the web were directly obtained from the same coil used to manufacture the specimen in this series. The material of the top plate was taken from the middle section of the flange between the longitudinal stiffeners after the cold-forming process. All materials were tested in the longitudinal, i.e. rolling, direction which is also the principal loading direction of the actual panel in bending, and testing of material transverse to the rolling direction was considered to be not necessary. The coated material of the base panel (G550) and the web (G350) was tested in its original coated state as well as in its uncoated state. However, the properties of the coated material, with values based on the base metal thickness (b.m.t.), were then used for further calculations. A detailed description of the test method and all test results can be found in Appendix A. The average results for the different materials used in this test series are listed in Table 8-2.
Table 8-2. Average tensile coupon test results

<table>
<thead>
<tr>
<th>Coupon</th>
<th>Coating</th>
<th>(t_c) (mm)</th>
<th>(t_b) (mm)</th>
<th>(f_{t*}) (MPa)</th>
<th>(\varepsilon_{0.2*}) (%)</th>
<th>(f_{u*}) (MPa)</th>
<th>(E^*) (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Top plate material (BLACKFORM)</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BF/2.5</td>
<td>uncoated</td>
<td>n/a</td>
<td>2.52</td>
<td>295</td>
<td>0.39</td>
<td>411</td>
<td>218.5</td>
</tr>
<tr>
<td>BF/1.6</td>
<td>uncoated</td>
<td>n/a</td>
<td>1.56</td>
<td>310</td>
<td>0.39</td>
<td>386</td>
<td>224.1</td>
</tr>
<tr>
<td>HA250/1.85</td>
<td>uncoated</td>
<td>n/a</td>
<td>1.84</td>
<td>261</td>
<td>0.37</td>
<td>354</td>
<td>225.3</td>
</tr>
<tr>
<td>HA70T/1.9</td>
<td>uncoated</td>
<td>n/a</td>
<td>1.90</td>
<td>488</td>
<td>0.49</td>
<td>498</td>
<td>204.0</td>
</tr>
<tr>
<td><strong>Web material (G350)</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>G350/0.9</td>
<td>coated</td>
<td>0.94</td>
<td>0.89</td>
<td>391</td>
<td>0.43</td>
<td>487</td>
<td>224.0</td>
</tr>
<tr>
<td></td>
<td>uncoated</td>
<td>n/a</td>
<td>0.89</td>
<td>389</td>
<td>0.43</td>
<td>482</td>
<td>211.4</td>
</tr>
<tr>
<td>G350/0.7</td>
<td>coated</td>
<td>0.75</td>
<td>0.69</td>
<td>391</td>
<td>0.42</td>
<td>485</td>
<td>228.2</td>
</tr>
<tr>
<td></td>
<td>uncoated</td>
<td>n/a</td>
<td>0.69</td>
<td>382</td>
<td>0.44</td>
<td>469</td>
<td>206.9</td>
</tr>
<tr>
<td><strong>Base panel material (G550)</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>G550/1.00-1</td>
<td>coated</td>
<td>1.07</td>
<td>0.99</td>
<td>677</td>
<td>0.54</td>
<td>687</td>
<td>232.9</td>
</tr>
<tr>
<td></td>
<td>uncoated</td>
<td>n/a</td>
<td>0.99</td>
<td>661</td>
<td>0.55</td>
<td>669</td>
<td>224.2</td>
</tr>
</tbody>
</table>

* values based on b.m.t.

8.3.4 Modifications prior testing

Further modifications of the panels were necessary prior to testing and were made at the Structural Engineering workshop of the University of Western Sydney, Australia:

- During the course of testing, it was soon realized that it was not possible to distinguish if the final top plate buckling failure was caused by an initial pull-out failure of the TOX connectors or actually due to buckling or yielding of the top plate. To eliminate this uncertainty, the top plate TOX connectors between the loading points were replaced with M5 bolts for the remaining majority of test panels. Another reason for replacing the TOX connectors with bolts was the fact that the pull-out strength of TOX connectors is mainly dependent on the size of the undercut, which can be controlled by using different types of TOX manufacturing tool sets. To ensure that failure would be in the plate panel, high-strength bolts (Grade 12.9) with nuts as their counterparts were used to simulate a TOX connection which would not fail in tension and therefore provided the best possible TOX-type connection. In doing so, an upper limit of the ultimate moment capacity can be established; noting that the usage of TOX connectors instead of bolts would possibly lead to slightly poorer results. Holes equal to the punch diameter of the TOX, i.e. 5 mm, were drilled through the press-joints, leaving the rim on the die side intact to place the nut on. This procedure and the use of bolts with flat head socket caps (as seen in Figure 8-11) ensured that the bolt connection would behave like a TOX joint without clamping the top plate and the upper web flange adjacent to the connection.

- The original test program considered only connector spacings up to 140 mm which was regarded as the upper limit in an actual application. Replacing the TOX joints with bolts revealed that larger spacings were necessary to possibly see an effect of the spacing. Larger spacings were achieved by removing unwanted connectors with an 8 mm drill bit and were therefore multiples of the initial connector spacing. As a result of using the TOX machine, the initial TOX arrangement was staggered. However, the stagger was less evident for some panels due to the increased spacing. Table 8-3 provides a list of all tested specimens with the location of the connectors near midspan. The offset in the east and west direction was measured relative to the centre line of each panel.
- For the close-range photogrammetry, retro-reflective targets with a diameter of 0.25 inch (about 6.35 mm) were glued onto the surface of the top plate at a nominal spacing of 1.00 inch (about 25.4 mm), as seen in Figure 8-6. Typically, three lines of targets were applied onto the plate between the longitudinal stiffeners (middle plate element), and two lines onto each edge plate element (north or south), as illustrated in Figure 8-5.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Spacing (mm)</th>
<th>North-East (mm)</th>
<th>North-West (mm)</th>
<th>South-East (mm)</th>
<th>South-West (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>90250B70-70T</td>
<td>70 (TOX)</td>
<td>30</td>
<td>70</td>
<td>90</td>
<td>10</td>
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<tr>
<td>90250B70-100</td>
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<td>48</td>
<td>14</td>
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</tr>
<tr>
<td>90250B70-120T</td>
<td>120 (TOX)</td>
<td>87</td>
<td>33</td>
<td>15</td>
<td>105</td>
</tr>
<tr>
<td>90250B70-140</td>
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<td>100</td>
<td>40</td>
<td>121</td>
<td>19</td>
</tr>
<tr>
<td>90250B70-200</td>
<td>200</td>
<td>44</td>
<td>156</td>
<td>14</td>
<td>186</td>
</tr>
<tr>
<td>90250B70-280</td>
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<td>112</td>
<td>168</td>
<td>131</td>
<td>149</td>
</tr>
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<td>52</td>
<td>346</td>
<td>14</td>
</tr>
<tr>
<td>90160B70-100T</td>
<td>100 (TOX)</td>
<td>43</td>
<td>57</td>
<td>2</td>
<td>98</td>
</tr>
<tr>
<td>90160B70-100</td>
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<td>89</td>
<td>11</td>
<td>48</td>
<td>52</td>
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<tr>
<td>90160B70-280</td>
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<td>93</td>
<td>204</td>
<td>76</td>
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<td>40</td>
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<td>145</td>
<td>55</td>
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<td>96</td>
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<td>68</td>
<td>90</td>
<td>110</td>
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<tr>
<td>140190H90-300</td>
<td>300</td>
<td>140</td>
<td>160</td>
<td>200</td>
<td>100</td>
</tr>
</tbody>
</table>

Figure 8-11. Bolt replacement (left; middle) and additional reinforcement (right)

- In using sections with higher webs (TD 140), the possibility of secondary failures, such as web buckling and lateral-distortional swaying, were more likely. This was accounted for by reinforcing the section over the support points. Therefore, additional diaphragms were attached to the panel and two 13 mm holes were drilled into the top plate in the vicinity of each support point to vertically insert 12 mm reinforcing bars which were then welded to the top plate, as shown in Figure 8-11.

- To simulate a lateral restraint from adjacent panels in an actual application, both ribs of the base panel were clamped to the 10 mm thick plate between the base panel and the roller supports. To prevent any possibility of a pull-out failure of the connectors in the relatively short shear span, additional TEK screws were placed in-between the connectors of the base panel near the loading points. In general, one TEK screw was put between connectors and no more than ten screws were placed on each side of the shear span for all specimens.
8.4 TEST RESULTS

8.4.1 General

In this test series, modifications were made to the specimens that suppressed any unwanted failure modes, in particular a shear failure of the connectors in the base panel and buckling of the corrugated webs. For the majority of test panels, the top plate TOX connectors between the loading points were replaced with M5 bolts to eliminate a preliminary pull-out failure of the TOX connectors, thus ensuring a top plate buckling or yielding failure. The modified panels therefore provide the best possible TOX-type connection solution which would not fail in tension. Specimens typically failed by forming an overall buckle across the entire width of the top plate near midspan after reaching the ultimate load, as seen in Figure 8-12. At the moment of buckling, the separation forces between the top plate and the upper web flanges increase rapidly due to second order large deformation effects, generating significant tension forces at the discrete location of the connectors. Eventually, the forces become so large that even the entire nut of the bolt replacement is completely pulled through the upper web flange, as seen on the right in Figure 8-12.

A different failure mode was observed for the TD 90 sections with a 1.60 mm thick top plate. At the ultimate load, the middle plate element between the longitudinal stiffeners buckled inwards, causing the edge plate elements to buckle upwards, as seen in Figure 8-13. After this local buckling occurred, an overall upwards buckling of the top plate adjacent to the local buckle was observed. In the specimen with TOX connectors, this overall buckling was sufficient to pull-out the TOX. In the specimens with the bolt replacement, the buckle occurred but was restrained by the bolts.
Table 6-6 contains ultimate load and ultimate moment results for all specimens, as well as vertical deflection at ultimate load. Deflections were measured at midspan and did not include self-weight of the panels.

### Table 8-4. Overview of test results

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Self-weight (kN/m)</th>
<th>Ultimate Jack Load (kN)</th>
<th>Moment at midspan (kNm)</th>
<th>Ultimate Jack Load (kNm)</th>
<th>Deflection at midspan (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>90250B70-70T</td>
<td>0.0900</td>
<td>16.90</td>
<td>0.2970</td>
<td>13.31</td>
<td>178.05</td>
</tr>
<tr>
<td>90250B70-100</td>
<td>0.0900</td>
<td>16.90</td>
<td>0.2970</td>
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<td>171.40</td>
</tr>
<tr>
<td>90250B70-120T</td>
<td>0.0900</td>
<td>16.42</td>
<td>0.2970</td>
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<td>162.62</td>
</tr>
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<td>90250B70-140</td>
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<td>16.87</td>
<td>0.2970</td>
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</tr>
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<td>16.70</td>
<td>0.2970</td>
<td>13.15</td>
<td>169.37</td>
</tr>
<tr>
<td>90250B70-280</td>
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<td>16.39</td>
<td>0.2970</td>
<td>12.91</td>
<td>163.58</td>
</tr>
<tr>
<td>90250B70-360</td>
<td>0.0900</td>
<td>16.15</td>
<td>0.2970</td>
<td>12.72</td>
<td>161.53</td>
</tr>
<tr>
<td>90160B70-70T</td>
<td>0.0733</td>
<td>9.45</td>
<td>0.2420</td>
<td>7.44</td>
<td>103.49</td>
</tr>
<tr>
<td>90160B70-100</td>
<td>0.0733</td>
<td>9.76</td>
<td>0.2420</td>
<td>7.69</td>
<td>110.98</td>
</tr>
<tr>
<td>90160B70-280</td>
<td>0.0733</td>
<td>9.34</td>
<td>0.2420</td>
<td>7.36</td>
<td>102.65</td>
</tr>
<tr>
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<td>0.2530</td>
<td>13.17</td>
<td>66.73</td>
</tr>
<tr>
<td>140185L70-200</td>
<td>0.0767</td>
<td>16.03</td>
<td>0.2530</td>
<td>12.62</td>
<td>63.40</td>
</tr>
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<td>140185L90-200</td>
<td>0.0783</td>
<td>16.91</td>
<td>0.2585</td>
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<tr>
<td>140185L90-300</td>
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<td>15.77</td>
<td>0.2585</td>
<td>12.42</td>
<td>58.76</td>
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<tr>
<td>140190H70-100</td>
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<td>19.51</td>
<td>0.2640</td>
<td>15.36</td>
<td>75.06</td>
</tr>
<tr>
<td>140190H70-200</td>
<td>0.0800</td>
<td>17.98</td>
<td>0.2640</td>
<td>14.16</td>
<td>66.98</td>
</tr>
<tr>
<td>140190H90-200</td>
<td>0.0817</td>
<td>19.15</td>
<td>0.2695</td>
<td>15.08</td>
<td>70.06</td>
</tr>
<tr>
<td>140190H90-300</td>
<td>0.0817</td>
<td>15.05</td>
<td>0.2695</td>
<td>11.85</td>
<td>51.33</td>
</tr>
</tbody>
</table>

Load deflection curves for all tested specimens with bolt-replacement are shown in Figure 8-14 to Figure 8-16. Specimens with the same section geometry show similar load-deflection behaviour, noting that slip in the shear span was unlikely to occur due to the close TOX spacing and the additional TEK screws in the base panel. Curves of specimens using top plate material of lower yield strength (BLACKFORM or HA250, as seen in Figure 8-14 and Figure 8-15) show a clear non-linear load-deflection relationship, indicating that the top plate material started to become plastic and was in the vicinity of yield at ultimate load. This behaviour cannot be observed with the high-strength top plate material (HA70T) which shows an almost linear load-deflection relationship up to ultimate load (see Figure 8-16), indicating that the material was still operating in the elastic range. Due to a slightly higher second moment of area, the stiffness of panels with a 0.90 mm web is marginally higher compared to panels with a 0.70 mm web. After reaching ultimate load, the specimens failed due to buckling of the top plate which resulted in a sudden drop of load until a state of equilibrium was established again. This sudden failure mode is typical for buckling failures, particularly in testing equipment with low stiffness. The tests were then stopped and the panels slowly unloaded.

### 8.4.2 Measurement of top plate deformations

Digital close-range photogrammetry was employed to measure the top plate deformations before and during flexural testing and involved photographing the object from several viewpoints at various loading epochs. Processing the photogrammetric information made it possible to precisely determine the three-dimensional spatial coordinates of each measuring point on the top plate (see Appendix F). The overall measurement error in vertical direction ranged from 0.037 mm to 0.045 mm, which is less than 3 % of the thinnest top plate thickness tested in this series.
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Figure 8-14. Load-Deflection curves (TD 90: 2.50 mm and 1.60 mm top plate)

Figure 8-15. Load-Deflection curves (TD 140: 1.85 mm top plate)

Figure 8-16. Load-Deflection curves (TD 140: 1.90 mm top plate)
This justifies the use of this technique to also determine initial imperfections since it is believed that the threshold of significance to buckling performance has a minimum imperfection amplitude of approximately 10% of the plate thickness (Rasmussen and Hancock, 1987). A more detailed description of this non-contact measurement technique and its use in this test series is given in Chapter 7.

During the testing of individual specimens, the last set of photographs was taken just prior failure of the top plate (typically at a load one or two percent below ultimate load $P_u$). It is believed that the deformations at this load level are similar to the deformations when failure occurred and are presented in the following. Figure 8-17 to Figure 8-33 display the observed top plate deformations for individual specimens where each figure consists of three illustrations. In the first illustration, all data points are plotted together, giving an overall picture of the global deformation of the top plate. Crosses on the 3D-contour plot indicate the actual measurement points. The second and third illustration display the local deviations for the longitudinal line with the most severe deformation in the edge plate elements (north or south) and the middle plate element between the longitudinal stiffeners (refer to Appendix F for the measurements of all longitudinal lines). The deviations were determined by subtracting the global curvature (curve of best fit) of the longitudinal line from the data points, as described in Chapter 7.6. A complete Fourier-based spectral analysis (Bernard et al., 1999) was considered to be not necessary as the buckled shapes were well defined and the buckling half-wavelengths as well as their amplitudes could be directly measured from the graphs. It should also be noted that in some cases the buckles were localized which would not have been picked by a Fourier analysis. The graphs also display the initial imperfections of the same longitudinal line which were obtained in a similar way by using photographs taken just prior to loading of the panels. Comparing initial imperfections with the final buckled shape enables one to decide if imperfections were significant in the development of the final buckles. Imperfections were considered to be significant if the length of the waves were similar to the waves of the final buckles, the amplitudes were close to or larger than the threshold of significance, i.e. 10% of the base metal plate thickness $t_b$, and were in the same direction as the buckles. For this reason, the graphs contain two horizontal lines ($\pm 0.1 t_b$) representing the threshold of significance to buckling performance. The second graph also contains the location of individual connectors to investigate a potential correlation between connector spacing and buckling wavelength.
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**TD90: 2.50 mm top plate, 0.70 mm webs, 100 mm spacing**

*Figure 8-17. Top plate deformation prior buckling (Specimen 90250B70-100)*
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TD90: 2.50 mm top plate, 0.70 mm webs, 120 mm spacing (TOX)

Figure 8-18. Top plate deformation prior buckling (Specimen 90250B70-120T)
TD90: 2.50 mm top plate, 0.70 mm webs, 140 mm spacing

Figure 8-19. Top plate deformation prior buckling (Specimen 90250B70-140)
TD90: 2.50 mm top plate, 0.70 mm webs, 200 mm spacing

Figure 8-20. Top plate deformation prior buckling (Specimen 90250B70-200)
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TD90: 2.50 mm top plate, 0.70 mm webs, 280 mm spacing

Figure 8-21. Top plate deformation prior buckling (Specimen 90250B70-280)
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TD90: 2.50 mm top plate, 0.70 mm webs, 360 mm spacing

Figure 8-22. Top plate deformation prior buckling (Specimen 90250B70-360)
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TD90: 1.60 mm top plate, 0.70 mm webs, 100 mm spacing (TOX)

Figure 8-23. Top plate deformation prior buckling (Specimen 90160B70-100T)
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TD90: 1.60 mm top plate, 0.70 mm webs, 100 mm spacing

Figure 8-24. Top plate deformation prior buckling (Specimen 90160B70-100)
TD90: 1.60 mm top plate, 0.70 mm webs, 280 mm spacing

Figure 8-25. Top plate deformation prior buckling (Specimen 90160B70-280)
Figure 8-26. Top plate deformation prior buckling (Specimen 140185L70-100)
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TD140: 1.85 mm top plate, 0.70 mm webs, 200 mm spacing

Figure 8-27. Top plate deformation prior buckling (Specimen 140185L70-200)
TD140: 1.85 mm top plate, 0.90 mm webs, 200 mm spacing

Figure 8-28. Top plate deformation prior buckling (Specimen 140185L90-200)
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TD140: 1.85 mm top plate, 0.90 mm webs, 200 mm spacing

Figure 8-29. Top plate deformation prior buckling (Specimen 140185L90-300)
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TD140: 1.90 mm top plate, 0.70 mm webs, 100 mm spacing

Figure 8-30. Top plate deformation prior buckling (Specimen 140190H70-100)
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TD140: 1.90 mm top plate, 0.70 mm webs, 200 mm spacing

Figure 8-31. Top plate deformation prior buckling (Specimen 140190H70-200)
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TD140: 1.90 mm top plate, 0.90 mm webs, 200 mm spacing

Figure 8-32. Top plate deformation prior buckling (Specimen 140190H90-200)
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TD140: 1.90 mm top plate, 0.90 mm webs, 300 mm spacing

Figure 8-33. Top plate deformation prior buckling (Specimen 140190H90-300)
A summary of the buckling half-wavelengths observed in Figure 8-17 to Figure 8-33 are given in Table 8-5 to Table 8-8. It can be seen that the buckling behaviour for the edge plate element (south or north) can differ from the middle plate element between the longitudinal stiffeners. The degree of wave correlation between the edge and middle plate elements is therefore categorized in the tables from none to strong. No correlation implies no interrelation between the two plate elements and is mostly accompanied by different buckling half-wavelengths (e.g. 90250B70-200). A strong correlation between the plate elements is given for very similar buckling characteristics such as similar buckling half-wavelengths, patterns and amplitudes (e.g. 140190H90-300). Correlations of medium magnitude typically have similar buckling characteristics, but only for the buckles with the largest amplitude, and are mostly more dominant on one edge plate element (e.g. 140190H90-200). Naturally, there is no clear dividing line between the individual categories and many intermediate stages exist. Since the buckling shape of the top plate tends to be not regular over the entire constant moment region, the average buckling half-wavelength ($\lambda_{\text{Av}}$) as well as the maximum ($\lambda_{\text{Max}}$) is given.

Only for specimens with a relatively thin 1.60 mm top plate were two varying wave patterns recognized on the middle plate element between the longitudinal stiffeners, viz. a local buckling mode with a short buckling half-wavelength ($\lambda_{\text{Loc}}$) and a global mode with a rather long buckling half-wavelength ($\lambda_{\text{Glob}}$). Generally, no short buckling waves were observed for the edge plate elements.

Similar to the classification of the wave correlation, the correlation between connector spacing and buckling half-wavelength of the edge plate element can be categorized (the middle plate element was considered not to be directly affected by the connector spacing and therefore not categorized). No correlation could be observed for close connector spacings, e.g. 100 mm, indicating that the row of connectors act as a continuous line of support. A strong correlation was noticed for wider connector spacings (apart from specimen 90160B70-280 which failed due to local buckling). If a specimen also showed a strong wave correlation, including similar buckling amplitudes, between the edge and middle plate elements (e.g. 140190H90-300), the failure mode was similar to a column-like buckling of the entire top plate between discrete connectors. For other specimens with strong connector correlation, minor discontinuities of the buckled shape were observed (90250B70-200, 90250B70-280, and 90250B70-360). The discontinuities coincide with the locations of the drilled out TOX, suggesting that the remaining hole caused stress concentrations which resulted in localized yielding of the material (which was already close to overall yielding in compression) and the formation of a minor kink. This phenomenon could, however, not be observed to such an extent for other top plate thicknesses/grades where the connector spacing has been increased by removing TOX joints, as the top plate was not as close to overall yield. The tables also contain information about the significance of initial imperfections for the development of the final buckling shape. Since the buckling behaviour for the edge plate element can differ from the middle plate element, the assessment was carried out individually for each plate element. Imperfections were considered to be significant if the length of the waves were similar to the waves of the final buckles, the amplitudes close to 10% of the plate thickness, and orientated in the same direction as the buckles (e.g. middle plate element of specimen 140190H70-100). An additional star (*) indicates if the amplitude of any single imperfection exceeded the threshold of significance.
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Table 8-5. Observed buckling half-wavelengths (TD90: 2.50 mm top plate)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Edge</th>
<th>Middle</th>
<th>Wave</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\lambda_{av}$ (mm)</td>
<td>$\lambda_{max}$ (mm)</td>
<td>Imperfections</td>
</tr>
<tr>
<td>90250B70-100</td>
<td>165</td>
<td>190</td>
<td>not sig.</td>
</tr>
<tr>
<td>90250B70-120T</td>
<td>185</td>
<td>190</td>
<td>not sig.</td>
</tr>
<tr>
<td>90250B70-140</td>
<td>160</td>
<td>180</td>
<td>not sig.</td>
</tr>
<tr>
<td>90250B70-200</td>
<td>100</td>
<td>120</td>
<td>not sig.</td>
</tr>
<tr>
<td>90250B70-280</td>
<td>140</td>
<td>150</td>
<td>not sig.</td>
</tr>
<tr>
<td>90250B70-360</td>
<td>180</td>
<td>200</td>
<td>not sig.</td>
</tr>
</tbody>
</table>

Table 8-6. Observed buckling half-wavelengths (TD90: 1.60 mm top plate)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Edge</th>
<th>Middle</th>
<th>Wave</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\lambda_{loc}$ (mm)</td>
<td>$\lambda_{glob}$ (mm)</td>
<td>Imperfections</td>
</tr>
<tr>
<td>90160B70-100T</td>
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<td>indet.</td>
<td>not sig.</td>
</tr>
<tr>
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<td>n/a</td>
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</tr>
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<td>not sig.</td>
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</table>

Table 8-7. Observed buckling half-wavelengths (TD140: 1.85 mm top plate)

<table>
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</thead>
<tbody>
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</tr>
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<td>195</td>
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<td>250</td>
<td>signif.</td>
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Table 8-8. Observed buckling half-wavelengths (TD140: 1.90 mm top plate)

<table>
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<th>Middle</th>
<th>Wave</th>
</tr>
</thead>
<tbody>
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<td></td>
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</tr>
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<td>not sig.</td>
</tr>
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<td>not sig.</td>
</tr>
<tr>
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<td>280</td>
<td>290</td>
<td>signif.*</td>
</tr>
</tbody>
</table>

8.4.3 Strain measurements

Due to the closed cellular shape of the hybrid steel deck and the way the panels were manufactured, it was not possible to attach strain gauges on the inside surface of the top plate which would have compensated for buckling effects. However, to get a reasonable estimate of the stresses achieved in the top plate, additional strain gauges were attached to the upper and lower part of the corrugated web, as well as to the bottom of the base panel. Figure 8-34 to Figure 8-36 illustrate the measured strains at a moderate load (10 kN), as well as at ultimate load, with negative values indicating compression and positive values tension. It can be seen that the strain distribution over the depth of the section is not linear, and that the strain measurements at the upper and lower part of the web are generally less than in a linear distribution. The differences, which also exist at a moderate load of 10 kN, could be attributed to the corrugations of the web as longitudinal slip was unlikely at midspan and in a region of constant moment.
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Figure 8-34. Strain distribution at moderate and ultimate load (TD 90: $t_{TP}=2.50$ mm)

Figure 8-35. Strain distribution at moderate and ultimate load (TD 140: $t_{TP}=1.85$ mm)

Figure 8-36. Strain distribution at moderate and ultimate load (TD 140: $t_{TP}=1.90$ mm)
For comparison reasons, strains $\varepsilon$ on the outside surface of the top plate and of the base panel were calculated using linear-elastic theory and the applied moment at midspan $M$:

$$
\varepsilon = \frac{M y}{E I_p}
$$

where $E$ is the Young’s modulus, $I_p$ the second moment of area of the panel and $y$ the distance from the neutral axis. Section properties were obtained from a cross-section analysis of the average panel geometry for each section type using the computer program THIN-WALL (THIN-WALL, 1994). The model also considered the effect of web corrugations, and is described in more detail in Chapter 8.5.2.

The elastic distribution of strains and the position of the neutral axis are predicted reasonably well. The predictions indicate that specimens with a 2.50 mm top plate (BLACKFORM steel) reached compressive stresses of 300 MPa (0.15 % strain) which compares well to the actual yield stress of the material of 295 MPa. The scatter of the results for the top plate and the upper part of the web are likely due to local buckling. There was little scatter of the results for the base panel in tension, although elastic calculations seem to slightly overestimate the strains in the base panel, in particular at moderate loads. Similar predictions were found for specimens with a 1.85 mm and 1.90 mm top plate which did not reach yield.

### 8.4.4 Stress computation

Using linear-elastic theory and the section properties obtained from a cross-section analysis of the average panel geometry (see Chapter 8.5.2), the stress of the top plate at midspan can be calculated as followed:

$$
f = \frac{(M + M_0) y}{I_p}
$$

where $I_p$ is the second moment of area of the panel, $y$ the distance from the neutral axis to the top plate, $M$ the applied moment at midspan, and $M_0$ the moment due to self-weight. Table 8-9 gives a summary of the calculated stresses $f_u$ (at mid-surface of the plate elements) at ultimate load for all tested specimens, as well as the yield stress $f_y$ of the various top plate material obtained from tensile testing (refer to Table 8-2).

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Spacing</th>
<th>$f_y$* (MPa)</th>
<th>$f_u$ (MPa)</th>
<th>Specimen</th>
<th>Spacing</th>
<th>$f_y$* (MPa)</th>
<th>$f_u$ (MPa)</th>
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<tr>
<td>90250B70-70T</td>
<td>70 (TOX)</td>
<td>295</td>
<td>297.1</td>
<td>90160B70-100T</td>
<td>100</td>
<td>310</td>
<td>250.5</td>
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<td>280</td>
<td>310</td>
<td>247.6</td>
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<td>296.6</td>
<td>90160B70-200</td>
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<td>293.7</td>
</tr>
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<td>100</td>
<td>261</td>
<td>245.2</td>
<td>140190H70-100</td>
<td>100</td>
<td>488</td>
<td>276.7</td>
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<td>235.3</td>
<td>140190H70-200</td>
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<td>140190H90-300</td>
<td>300</td>
<td>488</td>
<td>208.2</td>
</tr>
</tbody>
</table>

* values obtained from tensile testing and based on b.m.t.
The relation between yield stress and ultimate stress is illustrated in Figure 8-37 and Figure 8-38.

From Figure 8-37 it can be seen that specimens with a 2.50 mm top plate and a close connector spacing have reached stresses in the top plate equivalent to the yield stress of the material. Increasing the connector spacing up to 360 mm had only a little effect on the ultimate stress of the top plate, and failure was more likely to be dominated by yielding rather than buckling of the material. Specimens tested with the original TOX connectors reached similar stresses at ultimate load compared to specimens with the bolt replacement indicating that prior to failure, the separation forces between the top plate and the web were not high enough to cause a preliminary pull-out failure of the TOX connectors. A similar observation was made for specimen 90160B70-100T. Specimens with a 1.60 mm top plate failed at stresses below yield due to local buckling of the middle plate element.
between the longitudinal stiffeners. Generally, this failure mode is independent of the connector spacing and a reduction in the panel load carrying capacity can only be expected if the failure mode changes (e.g., column-like buckling failure of the entire top plate due to an overly large connector spacing). This, however, could not be observed for spacings up to 280 mm.

Figure 8-38 shows a direct comparison (similar section geometries and material thicknesses) between panels with different top plate material grades. Panels with a low-strength material (HA250) and a close connector spacing reach stresses close to the yield stress of the material, whereas panels with a high-strength material (HA70T) reach stresses of only about half of the material yield stress. In the latter case, the performance of the panels is clearly dominated by buckling of the top plate which is also supported by the medium to strong buckling wave correlations observed with photogrammetry. Medium to medium/strong buckling wave correlations were also observed for the panels with a low-strength material, indicating that the failure was possibly more influence by buckling rather than yielding of the material. Doubling the connector spacing from 100 mm to 200 mm had only a minor effect on the load carrying capacity of the panels whereas a further increase in spacing to 300 mm showed some effect. This was more evident for the panel with a high-strength material (140190H90-300) which experienced a failure mode that was similar to a column-like buckling of the entire top plate between discrete connectors. Irrespective of the top plate material grade, an increase of the web thickness resulted in slightly higher ultimate stresses. Since the TOX connectors were replaced with bolts, the load carrying capacity of the panels was therefore not influenced by the actual connector strength (which varies for TOX joints and is dependent on the individual material thicknesses) and the increase in stress could be attributed to the higher stiffness of the upper web flanges acting as a flexible support of the top plate.

8.5 SIMPLIFIED DESIGN

8.5.1 General

Current design methods of cold-formed thin-walled structures (AISI-2001, 2003; AS/NZS 4600, 2005) consider the interaction between yield of the material and elastic buckling (local, distortional and/or lateral-torsional) of the members. These design standards contain the direct strength method (AISI-2001 Supp, 2004; AS/NZS 4600, 2005) which allows the elastic buckling stresses or stress resultants to be obtained directly from a rational elastic buckling analysis. The elastic buckling capacity of members in compression and bending can be very efficiently investigated by using the finite strip method (Cheung, 1976) which is a variant of the more general finite element method (Zienkiewicz, 1971). The finite strip method differs principally in that only prismatic members can be analysed which reflects on how the structure is subdivided for analysis (longitudinal strips instead of elements) and which displacement functions are used to describe the displacements. The finite strip method normally uses harmonic functions in the longitudinal direction whereas polynomial functions are used to describe the variation in the transverse direction (the finite element methods normally uses polynomial functions in both directions). The finite strip method is often described as ‘semi-analytical’ since the displacement functions used in longitudinal direction are similar to those
produced using analytical methods. A comprehensive review of the finite strip method can be found elsewhere (Cheung and Tham, 1997; Cheung and Tham, 2000).

The computer program THIN-WALL (Papangelis and Hancock, 1995; THIN-WALL, 1994) has been developed at the University of Sydney, Australia to perform a semi-analytical finite strip buckling analysis of thin-walled sections under compression and bending, which also calculates the section properties and stresses in thin-walled cross-sections of general geometry (open or closed). The finite strip buckling analysis is based on the theory presented by Cheung and can be expressed in the following matrix equation.

\[
[K][D] - \lambda [G][D] = 0
\]

where \([K]\) and \([G]\) are the stiffness and stability matrices of the thin-walled member and \(\lambda\) is the load factor against buckling under the initially assumed applied longitudinal stress. The analysis uses orthotropic plate theory (Timoshenko and Woinowsky-Krieger, 1959) and displacement fields to obtain the stiffness matrix \([K]\). The calculation of the stability matrix \([G]\) is based on the potential energy method (Cheung, 1976) and further extended by matrices for flexural displacements (Przemieniecki, 1973) and membrane displacements (Plank and Wittrick, 1974) which allows studying a wide range of buckling modes (e.g. local, distortional, flexural and flexural-torsional). The buckling modes are the eigenvectors of \([D]\) and obtained from \(\lambda\) values for which the determinant of the coefficients of \([D]\) vanishes. To solve this eigenvalue problem, the program THIN-WALL uses a direct eigenvalue routine (Hancock, 1974).

Although the structure being analysed can be of general geometry, it must be uniform in thickness in the longitudinal direction and simply supported at its ends as a result of the harmonic function used in the longitudinal direction. The buckling modes computed are therefore for a single buckling half-wavelength which then means that the next half-wavelength in an actual long member would be mirrored in the opposite direction to fulfil continuity. The longitudinal edges of the structure may be simply supported, clamped or free along the full length of the plate system. Each longitudinal strip of the structure is able to deform in its plane (membrane displacement) as well as out of its plane (flexural displacement) in a single half sine wave over the length being analysed. Longitudinal stresses are assumed to be uniform along the length of each strip, but can vary linearly from one nodal line to the other. This allows applying a range of longitudinal stress distributions, varying from pure compression to pure bending. Figure 8-39 shows an example of the longitudinal strip subdivision and the buckling (membrane and flexural) displacements of an edge stiffened plate (Hancock, 1998).

For the hybrid steel deck, there are a number of limitations to the use of the finite strip method as follows. While simply supported ends may not be a problem for the edge and middle plate elements individually as they are free to buckle up and down, the entire top plate can only buckle upwards as the upper web flanges allow this deformation but the corrugated web itself prevents a downwards overall buckle. This behaviour cannot be modelled.
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The method only allows continuous connection between the longitudinal strips and hence cannot model the discrete connection of the connectors in the hybrid deck. However, if the spacing of the connectors is relatively small compared to the buckling half-wavelength the connection could then be considered as being continuous. When a number of elements are connected together to form a cross-section, as for the hybrid deck, the analysis can give buckle shapes in which the elements penetrate each other. For these cases, the solutions should be neglected. Considering all these limitations and making the appropriate assumptions, the finite strip method has been used in the case of the hybrid steel deck to estimate the elastic buckling stresses or stress resultants as it is a simple tool for designers to use.

8.5.2 Computational model

An inevitable shortcoming in using the finite strip method is that the structure is subdivided in longitudinal strips of uniform thickness along their length which does not allow the modelling of discrete connectors with a defined spacing $s$. Observations made during the series of experimental tests indicated that only a portion of the upper web flanges contributed to the flexible support of the top plate. As can be seen in Figure 8-40, only the flange in the vicinity of the connector experienced major distortions which resulted in the formation of yield lines when failure occurred. The distortions of the web flange between the connectors were less significant and most likely caused by the major distortions near the connectors. This observation was utilized in the development of a suitable finite strip model that considered the influence of discrete connectors by using an effective thickness approach.

Figure 8-40. Upper web flange distortions observed during testing
It was assumed that the upper web flange provides flexible support of the top plate only over an effective length $L_{\text{eff}}$ whereas the remaining portion of the flange ($s - L_{\text{eff}}$) does not contribute at all. According to the observations, the load spreads linearly from the centre of the connector to the web-flange intersection at an angle of approximately 60 degrees. This is illustrated in Figure 8-41.

![Figure 8-41. Illustration of effective length concept](image)

Taking 20 mm as the distance $b_c$ from the web-flange intersection to the centre of the connector, the effective length $L_{\text{eff}}$ comes to 69.3 mm which agrees well with the observed distance seen in Figure 8-40, noting that the nominal wavelength of the web corrugations is 35 mm. Since the finite strip method does not allow alternating web thicknesses along the length of a member (the full web thickness $t_w$ over the effective length $L_{\text{eff}}$ and a negligible thickness for the remaining portion of the flange), a continuous effective web thickness $t_{\text{eff}}$ was used instead. The effective web thickness was calculated to give a continuous flexural stiffness over the connector spacing $s$ equivalent to the localized flexural stiffness over the effective length $L_{\text{eff}}$ using the actual web thickness $t_w$:

\[
I_{\text{eff}} = I_{F1} : : \frac{L_{\text{eff}}}{12} t_w^3 = \frac{s}{12} \frac{L_{\text{eff}}}{12} \tag{8-4}
\]

\[
t_{\text{eff}} = t_w \sqrt{\frac{L_{\text{eff}}}{s}} \tag{8-5}
\]

In the model, the upper web flange was rigidly connected to the top plate at the centre of the connector. This was regarded as conservative since it did not take into account any clamping effect of the bolt. The small flange-outstand between the centre of the connector and the unstiffened edge was not modelled at all as it had no effect on the stiffness. The finite strip model represented the entire cellular cross-section of the hybrid steel deck. The model was therefore useful in determining accurate section properties, including centroid and second moment of area, which were used to calculate stresses attained during testing (see Chapter 8.4.3 and 8.4.4).

A special feature of the software allows the consideration of the corrugated web in the model by increasing the transverse flexural stiffness and reducing the longitudinal membrane stiffness. In the finite strip buckling analysis, this is achieved by modifying the property matrix of each longitudinal strip and in the cross-section analysis by reducing the elastic modulus of the corrugated elements.
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(Papangelis and Hancock, 1999). Applying this feature to the model was important since the use of a thin and flat web would have caused premature buckling of the web which did not occur in testing. Although the software only offers the option of sinusoidal corrugations, the variance to the actual shape of the corrugations was considered to be negligible. With the corrugations not extending all the way to the upper and lower web-flange intersection, an element with a high stress gradient was located between the corrugated and the flat elements to allow for the sudden jump in longitudinal stress caused by the different element stiffnesses.

Figure 8-42 shows the cross-section of the finite strip model and the element variables assigned to the individual parts. Values of the variables depended on the cross-section type being considered. In the buckling analysis, a bending moment $M$ was applied to the cross-section.

![Figure 8-42. Element variables assigned to finite strip model](image)

8.5.3 Analysis results

Since the finite strip buckling analysis applies to one buckle half-wavelength only (which is a result of the simply supported ends), the analysis has to be repeated over a range of lengths to be able to distinguish between various buckling modes. In the case of the hybrid steel deck, buckling half-wavelengths between 20 mm and 10,000 mm were investigated and typically five characteristic buckling modes could be identified as seen in Figure 8-43 to Figure 8-46.

1. For very short half-wavelengths (30-50 mm), a local buckling mode can be observed. This mode involves deformation of the middle plate element of the top plate without movement of the fold line between the flat element and the longitudinal stiffeners on either side.

2. For half-wavelengths of around 100 mm, a flange-distortional buckling mode is typical. It is mainly characterized by a rotation of the edge plate elements of the top plate at the fold line between the flat element and the longitudinal stiffener. The rotation is dependent on the stiffness provided by the upper web and upper web flange which deform accordingly.

3. Increasing the half-wavelengths to around 400 mm gradually changes the buckling mode into an overall-distortional mode where the entire top plate experiences an overall translation. The translation is restrained by the stiffness of the web which also deforms accordingly. The lowest buckling stresses are typically observed for this buckling mode.
Figure 8-43. Buckling stress vs. buckling half-wavelength (TD 90: \( t_{fp}=2.50 \) mm)

Figure 8-44. Buckling stress vs. buckling half-wavelength (TD 90: \( t_{fp}=1.60 \) mm)
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Figure 8-45. Buckling stress vs. buckling half-wavelength (TD 140: $t_{TP}=1.85$ mm)

Figure 8-46. Buckling stress vs. buckling half-wavelength (TD 140: $t_{TP}=1.90$ mm)
4. For long half-wavelengths (about 4,000 mm), the buckling mode changes over into a torsional/distortional mode of the entire member.

5. A further increase of the half-wavelength (10,000 mm) results in a pure lateral buckling mode where the entire member moves laterally without distortion or twist of the cross-section.

The graphs in Figure 8-43 to Figure 8-46 show the variation of the maximum (elastic) buckling stress of the top plate with the buckle half-wavelength for various section types and connector spacings of the hybrid steel deck and indicate the characteristic buckling modes at particular half-wavelengths which are displayed above the graph. The graphs also include the buckling performance of the top plate by itself under pure compression (TP Buckling) which is very similar to the buckling performance of the entire section for short buckle half-wavelengths (up to 150 mm). For comparison reasons, the Euler column-buckling curve of the overall top plate (TP Column Buckling), expressed in equation (8-6), is also presented and blends into the TP Buckling curve at buckle half-wavelengths of around 400 mm to 500 mm, as would be expected.

\[
 f_{Euler} = \frac{\pi^2 EI_{TP}}{I^2 A_{TP}} \quad (8-6)
\]

The diagram given in Figure 8-47 shows the relationship between the non-dimensional ultimate strength of the top plate \( f_u/f_y \) and the non-dimensional plate "slenderness" expressed as \( \sqrt{f_y/f_{ol}} \), where \( f_{ol} \) and \( f_y \) are the elastic buckling stress and the yield stress of the top plate, respectively. The ultimate strength \( f_u \) was calculated from the maximum moment obtained from the tests and the elastic section properties (see Chapter 8.4.4). The yield stress \( f_y \) of each material was obtained from tensile coupon tests and the elastic buckling stress \( f_{ol} \) from the finite strip buckling analysis as described before. The elastic buckling stress for individual section types and connector spacings was generally taken as the point with minimum buckling stress (usually the minimum point for the overall-distortional buckling mode). Since it is not possible to model the effect of discrete connectors in a finite strip analysis, the potential buckling of the top plate in-between the connectors had to be checked as well. For this reason, each minimum point was compared to the buckling stresses obtained from the buckling performance of the top plate by itself (TP Buckling) at buckling half-wavelengths equivalent to the connector spacing. For cases where the TP buckling stresses were lower, it was assumed the entire top plate would buckle between the discrete connectors and the points were shifted on the diagram as indicated by the dashed arrow.

Shown in Figure 8-47 is the design curve obtained from the Australian Cold-Formed Steel Standard (AS/NZS 4600, 2005) using the direct strength method for distortional buckling which is essentially the Winter formula for stiffened compression elements (Winter, 1947). Also shown is the design curve from the Australian Steel Structure Standard (AS 4100, 1998) for a cold-formed plate in uniform compression where both longitudinal edges are simply supported.
This design curve is based on the Von Karman effective width approach (Von Karman et al., 1932) which can be also expressed as

\[ \frac{b_e}{b} = \frac{\alpha}{\sqrt{f_y/f_{el}}} \]  

(8-7)

where \( \alpha \) reflects the influence of initial imperfections and residual stresses. For a “perfect” plate with no initial imperfections and no residual stresses the value of \( \alpha \) is 1.0 (see curve for elastic buckling) whereas for the plate condition above, the value is 0.74. Trahair and Bradford suggest a value of 0.65 for a similar hot-rolled plate (Trahair and Bradford, 1988) and this design curve is also shown in the diagram.

\[ \begin{array}{c|c|c|c|c|c}
\hline
\text{TP Buckling} & 90250B70 & 90160B70 & 140185L70 & 140185L90 & 140190L70 & 140190L90 \\
\hline
0.2 & 0.6 & 1.0 & 1.4 & 1.8 \\
\hline
\end{array} \]

\[ \begin{array}{c|c|c|c|c|c}
\hline
\sqrt{f_y/f_{el}} & 0.2 & 0.6 & 1.0 & 1.4 & 1.8 \\
\hline
\text{Yielding} & 0.6 & 1.0 & 1.4 & 1.8 \\
\hline
\end{array} \]

**Figure 8-47. Interaction of plate strength with plate buckling using finite strip buckling analysis**

It can be seen that while these design curves predict the general trend of the test results, none of the curves closely fits the data. There are a number of possible reasons for this which are related to the method of determining the elastic buckling stress \( f_{el} \) for the top plate.

In the finite strip model, the discrete and staggered connectors were substituted by a continuous support. This simplification involved the determination of an effective length \( L_{eff} \) which represents the sphere of influence that the discrete connector has on the upper web flange. The web flange with the effective length was then ‘smeared’ into a continuous element of reduced thickness \( t_{eff} \) by equating their flexural stiffnesses. Depending on the connector spacing, the reduction in the upper web flange thickness was between 57 % and 89 % of the original thickness. The use of a smeared continuous connection along the line of the connectors becomes even more problematic the wider the spacing of the discrete connectors becomes as buckling deformations between discrete connectors are, in reality, not prevented.
Another significant effect on the elastic buckling stress comes from the modelling of the web and the upper web flange. The model assumes that the web flange is rigidly clamped to the top plate at the centre of the connector. Although the TOX connectors were replaced with bolts, it is uncertain if the connection was able to rigidly clamp the web flange to the top plate and therefore did not allow any rotation of the web flange at the joint. Due to the manufacturing process, the distance \( b_c \) from the web-flange intersection to the centre of the connector can slightly vary along the length of the member, as can the position of the connector on the top plate. In addition, it is arguable if the length of the web flange in the model should be taken from the centre of the connector or from the outer radius of the rim of the TOX, as can be seen in Figure 8-11. Calculations have shown that the finite strip model reacts quite sensitively to minor changes made to the geometry of the upper web flange. For instance, a relatively minor change of the distance \( b_c \) of only 5 mm can result in a 20% to 30% change of the elastic buckling stress \( f_{ol} \) (as well as of the buckling half-wavelength \( \lambda \)) which can shift the test points by about 15% to 20% along the plate slenderness axis.

In the finite strip model, values of buckling stress are obtained for a range of buckling half-wavelengths. In plotting the test results, the minimum buckling stress was used and the corresponding buckling half-wavelength \( \lambda_{FSM} \) are shown in Table 8-10. In the observations of the buckle shapes, it was found that the average (for the TD 90 specimen with a 1.6 mm top plate the average global) buckling half-wavelength for the edge plate element \( \lambda_{Av.\,Edge} \) could differ from that for the middle plate element \( \lambda_{Av.\,Middle} \). In the model however, the modelling of two different buckling half-wavelengths that occur concurrently is not possible. For example, \( \lambda_{Av.\,Edge} \) for specimen 90250B70-100 is 165 mm, whereas \( \lambda_{Av.\,Middle} \) is 215 mm and \( \lambda_{FSM} \) 300 mm. In general, it can be seen that the buckling half-wavelengths obtained by the model are higher than the observed value (see Table 8-10). This indicates that the model with continuous connection is not able to accurately predict the buckling behaviour of the stiffened top plate with discrete connections to an unstiffened web flange element.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>( \lambda_{Av.,Edge} ) (mm)</th>
<th>( \lambda_{Av.,Middle} ) (mm)</th>
<th>( \lambda_{FSM} ) (mm)</th>
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8.5.4 Elastic member buckling analysis

To overcome the shortcomings of modelling the hybrid steel deck with a continuous support and considering the length of the member to be only one buckling half-wavelength, a two-dimensional elastic buckling analysis of the top plate was performed using the commercial software package MICROSTRAN (MICROSTRAN V7.0, 1999). The model comprised the top plate member under uniform compression with discrete, flexible support members joined to it which is essentially a beam
on elastic foundation. The top plate member had an overall length of 1.9 m, matching the length of the constant moment region of the test apparatus, and was fully restrained at one end whereas the other end allowed longitudinal movement but restrained all remaining degrees of freedom. The section properties of the top plate member were obtained from a cross-sectional analysis using the program THIN-WALL (THIN-WALL, 1994) and the nominal geometry of the top plate. The support members were chosen to be a nominal 500 mm long and hinged at the base. To allow unrestricted rotation of the top plate member, hinges were used to join the top plate member to the other end of the support member (basically a hinged column). The support members were equally distributed along the length of the top plate member at a constant spacing \( s \) matching the positions of the connectors in the top plate. The axial stiffness \( EA/l \) of the support members was set to be equal to the stiffness of the support given by the web in the vicinity of the discrete connector (noting that each member represents two discrete connectors) and was obtained by analysing the load-deformation response of the web. The load-deformation response is the spring stiffness \( k \) of the flexible support which can also be expressed in terms of an equivalent cross-sectional area \( A_{eq} \) for the support members:

\[
\frac{P}{\delta} = k = \frac{EA_{eq}}{l} \Rightarrow A_{eq} = \frac{Pl}{\delta E}
\]  

This web model was based on the same geometry used for the finite strip model and had different properties for \( t_x \) and \( I_z \) to account for the effect of the corrugated web (Papangelis and Hancock, 1999). The same effective length concept \( (L_{eff} = 2 \tan 60^\circ b_c) \), as discussed in Chapter 8.5.2, was employed to model the effect of a continuous web flange on the individual support members. In the web model, the web flange was clamped at the position of the applied load, but allowed vertical movement, simulating the bolted connection.

Although the software allows considering “tension-only” members in a linear elastic buckling analysis, such members cannot provide an elastic support in one direction (upwards) while providing a very stiff support in the other direction which would prevent any buckling modes exhibiting a downwards buckle of the entire top plate (impossible in reality due to web restraint). Instead, all possible buckling modes were permitted in the analysis, but only buckling shapes that were plausible under the given boundary conditions were considered to give the correct critical load factors. These shapes had to be tangential to the horizontal at locations of zero buckling deformation such that any downwards buckle could be “mirrored” as shown in Figure 8-48. It should be noted that an exact tangent at the turning point of the buckled shape could not always be obtained similar to Figure 8-48, in particular for wider connector spacings. However, the “mirroring” process was considered adequate for practical purposes.

Generally, a total of three overall buckles developed along the length of the top plate member, with a buckling half-wavelength \( \lambda \) of approximately 300 mm. The associated critical load factor was used to calculate the elastic buckling stress \( f_{ol} \) of the top plate member. All results were then plotted on the diagram given in Figure 8-49 that shows the relationship between the non-dimensional ultimate strength of the top plate \( (f_d/f_y) \) and the non-dimensional plate “slenderness” expressed as \( \sqrt{f_y/f_{ol}} \).
Figure 8-48. Possible buckling mode when middle buckle “mirrored”

The diagram also shows the design curves obtained from the Australian Cold-Formed Steel Standard (AS/NZS 4600, 2005), the Australian Steel Structure Standard (AS 4100, 1998) and a design curve suggested by Trahair and Bradford (Trahair and Bradford, 1988).

Figure 8-49. Interaction of plate strength with plate buckling using member buckling analysis

It can be seen that none of the design curves closely fits the data. There are a number of possible reasons for this, as follows. The top plate is modelled as a line element which does not allow for any local or distortional buckling modes other than overall plate buckling. The buckling behaviour is sensitive to the stiffness of the support members which model the behaviour of the discrete connection and the support provided by the upper web flange and the corrugated web. For example, changing the connection of the web to the top plate from clamped to pinned can reduce the buckling stress of the top plate by up to 60 % and would shift the test points by about 55 % along the plate slenderness axis. In addition, the buckling mode also changed from three full buckles within the length of the top plate.
to two full buckles. In modelling the support stiffness, the same simplification, as for the finite strip
model, was also made for the sphere of influence that the discrete connector has on the upper web
flange and hence could effect the assessment of the stiffness of the restraint. As for the finite strip
method, the stiffness was also sensitive to minor changes in the geometry of the upper web flange.

8.5.5 Conclusion

A series of flexural tests have been carried out to study the buckling behaviour of the
longitudinally stiffened and discretely connected top plate of the hybrid steel deck (for most tests, the
original TOX joints were replaced with bolts to eliminate the possibility of an initial pull-out failure of
the TOX connectors). Different top plate geometries/thicknesses were investigated in regards to the
effect of varying connector spacings. Also, the influence of different material grades and web
thicknesses were studied. Digital close-range photogrammetry was employed to measure the top plate
deformations before and during flexural testing. It was observed that the buckling behaviour for the
edge plate elements can differ from the middle plate element between the longitudinal stiffeners and
various degrees of wave correlations existed. In particular for wider connector spacings, a strong
correlation between connector spacing and buckling half-wavelength of the edge plate element was
recognized. Initial imperfections did not seem to have a regular pattern that could be considered to
have an influence on the buckling of the top plate except for a few significant cases. In addition, the
magnitude of the imperfections was small compared to the plate thickness.

Typically specimens failed by forming an overall buckle across the entire width of the top plate
near midspan after reaching ultimate load which lead to a pull-through failure of the connector
through the upper web flanges. A different failure mode could be observed for top plates with a
relatively large width-to-thickness ratio where the middle plate element buckled inwards, causing the
dge plate elements to buckle upwards. Notwithstanding the different geometries and material grades
used for the top plates, it can be concluded that the reduction in plate strength was not significant (less
than 25 %) with increases in connector spacing (up to 360 %). Specimens with a low-strength top
plate material were able to reach compressive stresses close to yield of the material, noting that
ultimate stresses were somewhat lower for top plates with a relatively large width-to-thickness ratio
due to the different failure mode. A failure mode similar to a column-like buckling of the entire top
plate between discrete connectors could be observed for a specimen with high-strength top plate
material and a wide connector spacing of 300 mm. Irrespective of the top plate material grade, an
increase of the web thickness resulted in slightly higher ultimate stresses, and could be clearly
attributed to the higher stiffness of the upper web flanges acting as a flexible support of the top plate.

To predict the observed buckling behaviour of the top plate, simple design models using the finite
strip method and an elastic member buckling analysis have been established. Since the two-
dimensional finite strip model did not allow the modelling of discrete connectors, an effective
thickness approach was developed to consider the effect of different connector spacings. For buckling
half-wavelengths up to 10 m, five characteristic buckling modes were identified for the hybrid steel
deck in bending (local, flange-distortional, overall-distortional, torsional/distortional, and lateral). The
The elastic buckling stress of the top plate was generally taken as the point with minimum buckling stress unless the buckling stress of the top plate by itself, at a buckling half-wavelength equivalent to the connector spacing, was lower. The interaction of plate strength with plate buckling was then compared with various design curves. Although these design curves predict the general trend of the test results, none of the curves closely fit the data. This is mainly related to the method of determining the elastic buckling stress of the top plate as the finite strip model uses a smeared continuous connection along the line of the connectors and reacts quite sensitively to minor changes made to the geometry of the upper web flange. In addition, the modelling of two different buckling half-wavelengths that occur concurrently (as observed during testing) is not possible and generally led to higher buckling half-wavelengths than observed in the tests. Modelling the hybrid steel deck using an elastic member buckling analysis had the advantage of considering discrete connectors and member lengths longer than a single buckling half-wavelength. However, being a two-dimensional model, particular cross-sections of members could not be considered and were therefore reduced to line elements. The model comprised the top plate member under uniform compression with discrete, flexible support members joined to it. The axial stiffness of the support members was set to be equal to the stiffness of support given by the web in the vicinity of the discrete connector. The results of the buckling analysis (using the lowest plausible buckling mode) were then used to plot the interaction of plate strength with plate buckling. The comparison with various design curves showed that none of these curves closely fits the data. This can be explained by the fact that the buckling behaviour is quite sensitive to the stiffness of the support members due to multiple reasons and the top plate is modelled as a line element that allows only overall plate buckling and no other modes.

Assessing the results of the simple design models, it can be concluded that none of the models accurately predict the elastic buckling behaviour of the top plate. This is mainly attributed to the simplifications made to the two-dimensional models that were necessary to account for the discrete connectors as well as to the sensitivity of the models in regards to minor changes of the geometry. To achieve better results in predicting the buckling behaviour of the top plate, a costly three-dimensional finite element analysis has to be performed, which is not favourable for designers, and was outside the scope of this thesis. However, despite the shortcomings of the simple design models described above, the top plate buckling of the hybrid steel deck could be designed by using the minimum buckling stress obtained from a finite strip analysis and a design curve based on the Von Karman effective width approach (using an $\alpha$ value of 0.65 to account for the influence of initial imperfections and residual stresses).
Chapter 9. Conclusions and recommendations

CHAPTER 9
CONCLUSIONS AND RECOMMENDATIONS

9.1 GENERAL
A study of the strength and behaviour of a new and innovative hybrid composite steel deck that has the capability to span unpropped in excess of eight metres in the formwork stage and to control the vertical deflections due to the applied load, has been carried out experimentally and analytically. Several potential failure modes of the closed cellular section that is formed by a number of elements connected to each other using intermittent discrete mechanical connectors were identified and significant aspects of the behaviour were more closely investigated. In particular, the basic behaviour of the unusual mechanical connectors (round TOX press-joints) and their effect on the hybrid steel deck has been studied. Testing has shown that the hybrid steel deck is a viable, effective and reliable system that can successfully be used in practical applications in commercial projects. Based on the results obtained from this research program, appropriate engineering models were developed to explain the observed behaviour and may be further used as the basis for design methods.

9.2 SUMMARY
A total of 42 single lap shear tests have been carried out to better understand the shear behaviour of round TOX press-joints when dissimilar material combinations, in regards to thickness and steel grades, are used for the hybrid steel deck (Chapter 3.1). As the shear strength of the joints is highly influenced by their forming process and closely related to the manufacturing tool sets used, a generalization is very difficult and conclusions are restricted to specimens with similar characteristics. Generally, four different failure modes could be identified, viz.: shear, pull-over, combined shear and pull-over, as well as bearing. It was found that higher shear load capacities could be achieved with larger TOX joint diameters as well as by increasing the punch side material thickness, noting that a lower strength material was always located on the die side of the joint. In contrast, increasing the thickness of the die side material generally resulted in a reduction of the ultimate shear load with the
exception when very thick punch side material was used. The joints exhibit a relatively high loading stiffness with an approximately linear load-deformation curve up to about two thirds of the ultimate load. Tests on multiple fasteners showed that the shear load capacity was close to the sum of the individual connector capacity despite the limited deformation capacity of the press-joints.

A further 58 tensile coupons (48 incorporating round TOX press-joints) have been tested to investigate the influence of the clinch connectors on the performance of sheet steel, in particular of high-strength steel (G550), and to determine how much the extruded material of the joint contributes to the ultimate strength of the coupons (Chapter 3.2). The tests have demonstrated that the extruded material is able to carry some of the applied stress and behave better than an actual hole, with the same inside diameter as the joint, which would carry zero stress. The behaviour for G300 material is comparable to a solid section whereas the actual tensile strength for G550 material is less than the actual tensile strength when based on a solid section. However, the effect of the press-joint on the ultimate tensile strength may be ignored if the width of the sheet steel is relatively high compared to the diameter of the joint.

In an extensive series of 40 small-scale tests, the behaviour of the TOX press-joints used in the hybrid steel deck has been studied in combined shear and tension (Chapter 5). A sophisticated, fully-adjustable test rig was specially developed to test the connector strength of any section height at various loading angles. Four different material combinations were under investigation resulting in the highest failure loads for pure shear (90 degree). Combinations with thicker punch side material achieved higher ultimate loads in pure shear but not necessarily in pure tension (0 degree). Depending on the material combination, significant load reductions up to 85% were registered in pure tension compared to the values in pure shear, partly due to the distinct shape of the cellular section of the hybrid steel deck. It was also found that changes in the loading angle were most significant between 60 degree and 90 degree. Based on these results, interactive relationships between shear and tension have been proposed where the approach of the Australian/New Zealand or British Steel Design Standard fits the test data more closely than the European Steel Design Standard.

Over 33 trial specimens of the hybrid steel deck with spans up to 8.46 m have been tested as part of a series of preliminary positive bending tests, allowing the development of an appropriate testing apparatus and associated measurement devices (Chapter 4). The tests gave vital information about the overall behaviour of the hybrid steel deck when the top plate is in compression and the base panel in tension. During the course of these preliminary tests, a range of possible failure modes (longitudinal shear failure of the TOX connectors in the base panel; top plate buckling under compression; distortion of the web openings; local buckling of the corrugated web over the support; and lateral-distortional sway of the top plate) could be observed, and several improvements to the hybrid steel deck have been undertaken to increase its performance (e.g. modified geometry of the top plate and introduction of a new, improved end-diaphragm). On the basis of these tests, decisions were made regarding the significant aspects of the behaviour that would be more closely investigated in the research of this thesis. As a result, the longitudinal shear failure of the TOX connectors in the base panel and the buckling failure of the top plate under compression were identified as the most dominant
failure modes and further investigated in an extensive series of flexural tests. The influence that web openings can have on the overall performance of the hybrid steel deck was investigated in a separate study and is not part of this thesis. Other failure modes, such as web buckling over the support and lateral-distortional sway of the top plate, appeared to be closely related to the effectiveness of the end-diaphragm and were considered to be of lesser importance since they can readily be eliminated by using appropriate end-diaphragms.

In this context, a numerical investigation (linear-elastic eigenmode extraction) into the effects that the new and improved open end-diaphragm would have on the local and global buckling of the cold-formed hybrid cellular deck was undertaken (Chapter 4.4). The parametric study covered three different section heights and spans from 1.5 m to 5.0 m. The results clearly indicated that the existence of the standard end-diaphragm enhances the overall performance of most of the investigated panels, provided that a failure of the TOX connectors did not occur. Lateral-distortional sway was the dominant failure mode for short to medium spans whereas longer spans resulted in top plate buckling. Increasing the height of the diaphragm had a positive effect on the maximum critical buckling stress, particularly for the larger section heights, but had an insignificant effect on the top plate buckling mode.

The effect of TOX connector spacing in the base panel on the behaviour of the hybrid steel deck has been studied in a programme of 18 laboratory full-scale tests (Chapter 6). The series included three different material combinations and two different panel versions, viz. a configuration with anchored ends and with free ends. Fracture of the base panel indicated that a state of complete shear connection was achieved whereas longitudinal shear failure of the TOX connectors occurred for panels with partial shear connection. Generally, higher loads could be observed with thicker base panel material whereas an increase in the web thickness slightly reduced the ultimate load. A distinct relation between ultimate load and connector spacing was also noted. Measurements of longitudinal slip showed a fundamentally different distribution along the length of the panel compared to the conventional Newmark theory used for composite steel-concrete beams. A simple computational model, based on the Vierendeel truss analogue, was developed to model the slip behaviour of the panels and could be also used to predict the deformation performance of the hybrid steel deck which is mainly influenced by the development of longitudinal slip. The investigation on a faulty TOX connector in a critical region indicated that maintaining a regular connector spacing is essential to the proper performance of the hybrid steel deck. Finally, the ultimate moment capacity of the tested panels could be well predicted by using a partial shear connection model in conjunction with elastic theory, noting that neglecting the effect of the web would lead to more conservative results.

The measurement of the out-of-plane distortions of the top plate played an important role in understanding the failure behaviour of the hybrid steel deck. Various measurement techniques were considered, but digital close-range photogrammetry has been shown to provide highly accurate and extensive data of initial imperfections and deformations, in particular shapes of local buckles, of the top plate during testing (Chapter 7). The technique gives the opportunity to survey any number of targets at desired positions and has no physical contact with the object. Although this measurement
method is highly suitable for buckling problems, in particular of thin-walled structures, its use in structural engineering applications has been limited so far and left to specialist users of the photogrammetric technique. However, advances in digital camera technology and the development of suitable photogrammetric software packages have made this technique more attractive to laypersons and researchers. In the application of the hybrid steel deck, it was possible to achieve highly accurate measurements up to 0.02 mm with an affordable measurement system which was fully implemented and operated by a non-specialist user of the photogrammetric technique.

In a series of 18 full scale flexural tests, the buckling behaviour of the longitudinally stiffened and discretely connected top plate of the hybrid steel deck has been studied (Chapter 8). For most tests, the original TOX joints were replaced with bolts to eliminate the possibility of an initial pull-out failure of the TOX connectors. Different top plate geometries/thicknesses were investigated in regards to the effect of varying connector spacings. Also the influence of different material grades and web thicknesses were studied. Digital close-range photogrammetry was employed to measure the top plate deformations before and during flexural testing. It was observed that the buckling behaviour for the edge plate elements can differ from the middle plate element between the longitudinal stiffeners and various degrees of wave correlations existed. In most cases, initial imperfections were not considered to have an influence on the buckling of the top plate. Typically specimens failed by forming an overall buckle across the entire width of the top plate near midspan after reaching ultimate load. This led to a pull-through failure of the connectors through the upper web flanges (a different failure mode could be observed for top plates with a relatively large width-to-thickness ratio). Notwithstanding the different geometries and material grades used for the top plates, increasing the connector spacing by up to 360% had no significant influence on the plate strength. Specimens with a low-strength top plate material were able to reach compressive stresses close to yield of the material. Irrespective of the top plate material grade, an increase of the web thickness resulted in slightly higher ultimate stresses, and could be clearly attributed to the higher stiffness of the upper web flanges acting as a flexible support of the top plate.

Simple design models using the finite strip method and an elastic member buckling analysis have been established to predict the observed buckling behaviour of the top plate. Since the two-dimensional finite strip model did not allow the modelling of discrete connectors, an effective thickness approach was developed to consider the effect of different connector spacings. The interaction of plate strength with plate buckling was then compared with various design curves for steel structures. Although these design curves predict the general trend of the test results, none of the curves closely fit the data. This is mainly related to the method of determining the elastic buckling stress of the top plate and the inability of the model to consider different buckling half-wavelengths that occur concurrently. To consider the discrete connectors of the hybrid steel deck, a model using an elastic member buckling analysis was developed. The results (using the lowest plausible buckling mode) were then used to plot the interaction of plate strength with plate buckling. This also showed that none of the design curves closely fits the data. This can be explained by the fact that the buckling behaviour is quite sensitive to the stiffness of the members representing the upper web flange and that
the top plate is reduced to a line element that allows only overall plate buckling. In conclusion, it can be said that none of the simple design models accurately predict the elastic buckling behaviour of the top plate. However, despite the shortcomings of the models, the top plate buckling of the hybrid steel deck could be designed by using the minimum buckling stress obtained from a finite strip analysis and a design curve based on the Von Karman effective width approach.

9.3 SUGGESTIONS FOR FURTHER STUDIES

The work conducted in this thesis on a new and innovative hybrid composite steel deck forms the basic understanding of its complex behaviour in the formwork stage. Although individual components have been tested to determine their mechanical resistance developed in hardened concrete, an experimental program of full-scale composite tests is recommended to verify these results and develop a better understanding of the hybrid steel deck in the composite stage. However, its use in practical applications has shown that the hybrid composite steel deck performs satisfactorily under the given circumstances. In regards to the formwork stage, several areas for further study have been identified and require additional research.

1. **TOX press-joints**

Tests on the behaviour of the round TOX press-joints have shown that the strength of the mechanical joints is highly influenced by their forming process (in particular the tool set used for manufacturing) and the materials used to form the joint. In the absence of adequate design rules to predict their shear or pull-out capacity, testing is required whenever the parameters (e.g. tool set, material thickness, material grade, etc.) have changed. The specially developed test rig can therefore be of great use to obtain the needed results.

2. **End-diaphragm**

Although the standard open end-diaphragm is suitable for most applications with a section height up to 140 mm, further work is required for taller sections. Also, further studies may be needed for cases where lateral-distortional sway becomes the dominant failure mode.

3. **Longitudinal shear of the TOX joints in the base panel**

Tests carried out in this thesis cover the normal cases for which the hybrid steel deck is used. However, for special cases (e.g. irregular point loading), testing with a loading regime that places greater demand on the shear redistribution between the TOX joints is recommended. Although the simple computational model is able to predict the deformation performance of the hybrid steel deck satisfactorily, there is room for improvement for a more general elastic model that automatically takes the non-linear stiffness of the connectors into account. For practical applications, the model should also consider the slip behaviour of the connectors in the top plate.

4. **Buckling of the top plate**

None of the proposed simple design models accurately predicted the elastic buckling behaviour of the top plate. To achieve better results, a more complex model (e.g. a three-dimensional finite element model) has to be established that is able to take account of the discrete connectors and the particular cross-section of the top plate. Since, in most cases, the TOX joints in the top plate were
replaced with bolts, the influence of the TOX joints on the top plate buckling behaviour (e.g. initial pull-out failure) needs to be further investigated. The pull-out capacity of the TOX joints should also be included in the computational model.

5. **Bending tests with continuous spans**
   Although the ultra long-spanning hybrid steel deck is primarily used in single span applications, bending tests with continuous spans are required to obtain data on the load-carrying capacity of the hybrid steel deck at an internal support where the top plate is in tension and the base panel in compression. Additional failure modes are likely to be observed.

6. **Base panel flange curling**
   Flange curling occurs in flanges of thin-walled sections undergoing flexure and both tensile and compression flanges are subjected to it. Curling of the base panel, particularly for very thin sections at large deflections, may cause significant transverse bending within the closed section and relatively large bending moments may occur at the connections. The associated prying may cause additional tensile forces in the connections, resulting in a premature pull-out failure of the TOX connectors. While this has not been a problem in current applications, this aspect needs further study.

7. **Shear behaviour**
   Tests regarding the shear strength of the hybrid steel deck, in particular the corrugated webs, are considered necessary for applications with relatively short spans or concentrated loading close to the supports. Emphasis should be given to webs with openings and sections of taller heights.

8. **Influence of web holing**
   Basic research on the influence that web openings can have on the overall performance of the hybrid steel deck has been carried out. However, more research is required to cover the numerous possibilities web holing offers (e.g. hole size, holing pattern, hole position, etc.).

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CONSUMMATUM EST!
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Appendix A
Tensile coupon tests

A.1 GENERAL

Tensile testing of coupons was carried out to determine the mechanical properties of the material used in each set of test specimens, following the guidelines of the Australian Standard (AS 1391, 1991). All tests were completed in the Structural Engineering workshop of the University of Western Sydney, Australia using an INSTRON universal testing machine (Model No. 6027) with friction grips to apply the loading (see Figure A-1). The machine was equipped with the standard 200 kN load cell and calibrated according to the NATA (National Association of Testing Authorities, Australia) specifications. During placement of the tensile coupon, special care was taken to centre the specimen in the grips by plumbing it with respect to the vertical to reduce the possibility of load eccentricity. After the tensile coupon had been aligned and securely gripped in the test machine, two standard 50 mm gauge-length INSTRON clip-on extensometers (Model No. 2630-100) were attached directly to the specimen on opposite sides at the central portion of the constant gauge length (see on the right of Figure A-1). Load, cross-head movement and strains were recorded digitally every second using the software package MERLIN installed on a personal computer. Tests were undertaken in displacement control with a constant cross-head speed in the elastic range. Table A-1 gives an overview of the cross-head speed and the approximate elastic strain rate of each material tested. Stress-strain curves were obtained from the recordings of load and strains (average value of both extensometers) and are presented in Figure A-3 to Figure A-18.

Table A-1. Overview of the cross-head speeds and the approximate elastic strain rates

<table>
<thead>
<tr>
<th>Material</th>
<th>Cross-head (mm/min)</th>
<th>El. strain rate (mm/mm/min)</th>
<th>Material</th>
<th>Cross-head (mm/min)</th>
<th>El. strain rate (mm/mm/min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>G550/1.00</td>
<td>0.1</td>
<td>0.3 E-3</td>
<td>BF/2.5</td>
<td>0.3</td>
<td>1.0 E-3</td>
</tr>
<tr>
<td>G550/0.75</td>
<td>0.1</td>
<td>0.5 E-3</td>
<td>BF/1.6</td>
<td>0.3</td>
<td>1.5 E-3</td>
</tr>
<tr>
<td>G350/0.90</td>
<td>0.5</td>
<td>3.0 E-3</td>
<td>G250/1.85</td>
<td>0.3</td>
<td>1.2 E-3</td>
</tr>
<tr>
<td>G350/0.70</td>
<td>0.5</td>
<td>3.0 E-3</td>
<td>HA70T/1.90</td>
<td>0.3</td>
<td>1.0 E-3</td>
</tr>
</tbody>
</table>
Appendix A. Tensile coupon tests

A 2

A.2 SPECIMEN AND MATERIAL DETAILS

The size of the tensile coupons was based on the Australian Standard methods for tensile testing of metals (AS 1391, 1991) following recommendations from a study undertaken at the University of Sydney, Australia (Rogers and Hancock, 1996). The Standard suggests a minimum transition radius of 12 mm for 12.5 mm wide test pieces with a rectangular cross-section to ensure that fracture occurs in the central region of the test specimen, noting that a greater radius may be needed for material of low ductility, such as G550. Studies undertaken at the University of Sydney have shown that G550 sheet steels are sensitive to material imperfections and stress concentrations (Maladakis and Ayoub, 1994) which can affect their tensile behaviour. Tensile coupons with a sharp radius may fail prematurely at the gauge/radius junction due to stress concentrations caused by the sudden increase in cross-sectional area. In a subsequent study, specimens with a more gradual change in cross-sectional area, i.e. with a radius \( r \) of 55 mm, were found to give good results (Rogers and Hancock, 1996). Figure A-2 shows the size and shape of the tensile coupons used in the following test series: combined shear-tension tests of TOX joints (Chapter 5), base panel shear tests (Chapter 6), and top plate buckling tests (Chapter 8).

A CNC milling machine (BRIDGEPORT VMC56022) was used to achieve accurate and consistent test specimens, in particular the required large radius of 55 mm (using a 20 mm high speed steel slot drill). A specially constructed jig held the 20x230 mm blanks in place which were previously guillotined to size. Material for the base panel (G550) and the web (G350) were directly obtained from the same coil used to manufacture the specimen in the corresponding test series, whereas the top plate material was taken from the middle section of the flange between the longitudinal stiffeners after the cold-forming process. The top plate material used in the combined shear-tension test series
Appendix A. Tensile coupon tests

(Chapter 5) was considered to be non-influential and properties were therefore not determined. Although it is well-known that the material properties of cold reduced steels are anisotropic (Dhalla and Winter, 1974), only specimens cut from the longitudinal (rolling) direction were tested. Testing of material transverse to the rolling direction was considered to be not of importance since loading in the longitudinal direction was the principal loading direction of the actual panels in bending.

The materials used to manufacture the hybrid steel deck test specimens were all supplied by BLUESCOPE STEEL Ltd., Australia. The material of the base panel (G550) was a hot-dipped zinc-coated structural steel with a spangled surface, a guaranteed minimum yield strength of 550 MPa and a total zinc coating mass of approximately 370 g/m$^2$ (ZINC HI-TEN G550-Z350). The material of the web (G350) was also a hot-dipped zinc-coated structural steel with a spangled surface, a guaranteed minimum yield strength of 350 MPa and a total zinc coating mass of approximately 370 g/m$^2$ (ZINCFORM G350-Z350). The following uncoated materials were used for the top plate. A hot-rolled low carbon steel with no guaranteed minimal yield strength (BLACKFORM steel). However, the yield strength is typically in the range of 250 MPa to 380 MPa. A hot-rolled structural steel with a guaranteed minimum yield strength of 250 MPa (HA250), as well as a hot-rolled and tempered rolled steel with a guaranteed minimum hardness of 70 HRB and a guaranteed minimum yield strength of 440 MPa (HA70T). A summary of the typical chemical composition of the steel base material is given in Table A-2.

<table>
<thead>
<tr>
<th>Material</th>
<th>Carbon (C)</th>
<th>Phosphorus (P)</th>
<th>Manganese (Mn)</th>
<th>Sulphur (S)</th>
<th>Silicon (Si)</th>
<th>Aluminium (Al)</th>
<th>Nitrogen (N)</th>
</tr>
</thead>
<tbody>
<tr>
<td>G550</td>
<td>0.035-0.07</td>
<td>0.00-0.02</td>
<td>0.20-0.30</td>
<td>0.00-0.02</td>
<td>0.00-0.02</td>
<td>0.02-0.07</td>
<td>0.00-0.008</td>
</tr>
<tr>
<td>G350</td>
<td>0.13-0.18</td>
<td>0.00-0.03</td>
<td>0.60-0.90</td>
<td>0.00-0.02</td>
<td>0.00-0.03</td>
<td>0.015-0.08</td>
<td>0.00-0.01</td>
</tr>
<tr>
<td>BF/2.5</td>
<td>0.08-0.12</td>
<td>0.01-0.02</td>
<td>0.35-0.50</td>
<td>0.01-0.02</td>
<td>0.005-0.01</td>
<td>0.02-0.05</td>
<td>0.001-0.004</td>
</tr>
<tr>
<td>BF/1.6</td>
<td>0.04-0.07</td>
<td>0.01-0.02</td>
<td>0.20-0.30</td>
<td>0.01-0.02</td>
<td>0.005-0.01</td>
<td>0.03-0.05</td>
<td>0.001-0.005</td>
</tr>
<tr>
<td>HA250</td>
<td>0.09-0.17</td>
<td>0.005-0.025</td>
<td>0.50-0.75</td>
<td>0.005-0.02</td>
<td>0.005-0.02</td>
<td>0.015-0.06</td>
<td></td>
</tr>
<tr>
<td>HA70T</td>
<td>0.04-0.07</td>
<td>0.01-0.02</td>
<td>0.20-0.30</td>
<td>0.01-0.02</td>
<td>0.005-0.01</td>
<td>0.03-0.05</td>
<td>0.001-0.005</td>
</tr>
</tbody>
</table>

The basic material properties, i.e. yield stress ($f_y$), ultimate strength ($f_u$), and Young’s modulus ($E$), for all the materials, were obtained through tensile testing. It is international practice to test zinc-coated sheet steel with the coating intact, but to calculate strength on base metal thickness since the contribution made by the zinc coating can be ignored for practical purposes (AS 1397, 2001). However, for coated G300 sheet steels, yield stress increases of over 10.7% and ultimate strength increases of over 8.5% have been reported (Willis, 1982). To quantify the influence of the metallic coating on the materials used for the hybrid steel deck (i.e. G550 and G350), coated as well as uncoated tensile coupons were tested.

Uncoated specimens were obtained by acid-etching milled coupons in a diluted hydrochloric acid bath (four parts acid and one part distilled water). Generally, the mechanical properties were based on the base metal thickness (b.m.t.) of the test specimen. A calibrated 25 mm micrometer (MITUTOYO Model No. 502863) was used to obtain the specimen thickness, viz. coated ($t_c$) and uncoated ($t_b$). The gauge width ($b$) was measured at different locations using a calibrated 200 mm vernier (MITUTOYO Model No. 500-172) and averaged.
A.3 TENSILE COUPON TEST RESULTS

All G550 sheets steels tested in this series yielded gradually with minimal strain hardening, whereas the G350 sheet steels displayed a sharp yield point, followed by yield elongation plateau, then a strain hardening region. A similar stress-strain behaviour was observed for the BLACKFORM and HA250 steels whereas the high-strength steel HA70T displayed continuous yielding with minimal strain hardening. Generally, all specimens fractured in the central region of the constant gauge length.

Yield stress ($f_y$) values for material displaying continuous yielding (i.e. G550 and HA70T) were calculated using the 0.2% strain offset method whereas an averaged stress value of the yield elongation plateau was used for the materials exhibiting discontinuous yielding (i.e. G350, BLACKFORM and HA250). The strain at 0.2% proof stress ($\varepsilon_{0.2}$) is the corresponding strain value of the material when using the 0.2% strain offset method. Young’s modulus of elasticity ($E$) was obtained from the slope of the linear region of the stress-strain curves of each specimen using the method of least squares.

A comparison between coated and uncoated tensile coupons of the same material revealed only a small contribution (up to 3.4%, but typically around 2.5% or less) of the metallic coating on the strength of the sheet steel materials. A small increase of the Young’s modulus was also noted for coated specimens. Generally, a much higher value than the commonly assumed value of 200 GPa was observed, with values up to 235 GPa for coated G550 steels. The measured values correspond well with other studies on coated and uncoated sheet steels (Mahendran, 1996).

For selected G550 sheet steels, an idealized material curve which was based on a modified Ramberg-Osgood curve (Bezkorovainy et al., 2003) was added to the actual stress-strain curve of the material, and is expressed in the following form

$$\varepsilon = \frac{\sigma}{E_0} + 0.002 \left( \frac{\sigma}{\sigma_{0.2}} \right)^n$$

where $E_0$ is the initial elastic modulus, $\sigma_{0.2}$ the 0.2% proof stress and $n$ a positive constant that defines the sharpness of the knee of the stress-strain curve. The values of the quantities ($E_0$, $\sigma_{0.2}$, $n$) for each material are given in Table A-3.

<table>
<thead>
<tr>
<th>Material</th>
<th>$E_0$ (GPa)</th>
<th>$\sigma_{0.2}$ (MPa)</th>
<th>$n$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.00 mm (G550), coated</td>
<td>233</td>
<td>677</td>
<td>45</td>
</tr>
<tr>
<td>1.00 mm (G550), uncoated</td>
<td>224</td>
<td>661</td>
<td>80</td>
</tr>
<tr>
<td>0.75 mm (G550), coated</td>
<td>235</td>
<td>697</td>
<td>20</td>
</tr>
<tr>
<td>0.75 mm (G550), uncoated</td>
<td>226</td>
<td>695</td>
<td>20</td>
</tr>
</tbody>
</table>
A.3.1 Results of combined shear-tension tests

Table A-4. Tensile coupon test results for coated base panel material (G550)

<table>
<thead>
<tr>
<th>Coupon</th>
<th>Coating</th>
<th>t_c (mm)</th>
<th>t_b (mm)</th>
<th>b (mm)</th>
<th>f_y* (MPa)</th>
<th>ε_{0.2}* (%)</th>
<th>f_u* (MPa)</th>
<th>E* (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>G550/1.00-1</td>
<td>coated</td>
<td>1.06</td>
<td>0.99</td>
<td>12.53</td>
<td>650</td>
<td>0.54</td>
<td>661</td>
<td>228.9</td>
</tr>
<tr>
<td>G550/1.00-2</td>
<td>coated</td>
<td>1.07</td>
<td>0.99</td>
<td>12.50</td>
<td>645</td>
<td>0.52</td>
<td>662</td>
<td>231.7</td>
</tr>
<tr>
<td>G550/1.00-3</td>
<td>coated</td>
<td>1.06</td>
<td>0.99</td>
<td>12.56</td>
<td>647</td>
<td>0.52</td>
<td>657</td>
<td>243.8</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td></td>
<td>1.06</td>
<td>0.99</td>
<td>12.53</td>
<td>647</td>
<td>0.53</td>
<td>660</td>
<td>234.8</td>
</tr>
<tr>
<td>G550/0.75-1</td>
<td>coated</td>
<td>0.81</td>
<td>0.74</td>
<td>12.53</td>
<td>712</td>
<td>0.55</td>
<td>715</td>
<td>245.5</td>
</tr>
<tr>
<td>G550/0.75-2</td>
<td>coated</td>
<td>0.82</td>
<td>0.74</td>
<td>12.50</td>
<td>703</td>
<td>0.55</td>
<td>705</td>
<td>240.7</td>
</tr>
<tr>
<td>G550/0.75-3</td>
<td>coated</td>
<td>0.82</td>
<td>0.74</td>
<td>12.51</td>
<td>715</td>
<td>0.55</td>
<td>718</td>
<td>235.9</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td></td>
<td>0.82</td>
<td>0.74</td>
<td>12.51</td>
<td>710</td>
<td>0.55</td>
<td>713</td>
<td>240.7</td>
</tr>
</tbody>
</table>

* values based on b.m.t.

Figure A-3. Stress-strain curve for 1.00 mm thick G550 steel (coated)

Figure A-4. Stress-strain curve for 0.75 mm thick G550 steel (coated)
### Table A-5. Tensile coupon test results for coated web material (G350)

<table>
<thead>
<tr>
<th>Coupon</th>
<th>Coating</th>
<th>$t_c$ (mm)</th>
<th>$t_o$ (mm)</th>
<th>$b$ (mm)</th>
<th>$f_y^*$ (MPa)</th>
<th>$\varepsilon_{0.2}^*$ (%)</th>
<th>$f_u^*$ (MPa)</th>
<th>$E^*$ (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>G350/0.9-1</td>
<td>coated</td>
<td>0.93</td>
<td>0.88</td>
<td>12.46</td>
<td>387</td>
<td>0.43</td>
<td>486</td>
<td>226.4</td>
</tr>
<tr>
<td>G350/0.9-2</td>
<td>coated</td>
<td>0.94</td>
<td>0.89</td>
<td>12.50</td>
<td>387</td>
<td>0.43</td>
<td>486</td>
<td>225.7</td>
</tr>
<tr>
<td>G350/0.9-3</td>
<td>coated</td>
<td>0.93</td>
<td>0.88</td>
<td>12.51</td>
<td>391</td>
<td>0.42</td>
<td>487</td>
<td>227.3</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td></td>
<td>0.93</td>
<td>0.88</td>
<td>12.49</td>
<td>388</td>
<td>0.43</td>
<td>486</td>
<td>226.5</td>
</tr>
<tr>
<td>G350/0.7-1</td>
<td>coated</td>
<td>0.76</td>
<td>0.70</td>
<td>12.50</td>
<td>413</td>
<td>0.43</td>
<td>511</td>
<td>226.6</td>
</tr>
<tr>
<td>G350/0.7-2</td>
<td>coated</td>
<td>0.76</td>
<td>0.70</td>
<td>12.49</td>
<td>411</td>
<td>0.44</td>
<td>510</td>
<td>223.4</td>
</tr>
<tr>
<td>G350/0.7-3</td>
<td>coated</td>
<td>0.75</td>
<td>0.69</td>
<td>12.46</td>
<td>412</td>
<td>0.44</td>
<td>509</td>
<td>217.3</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td></td>
<td>0.76</td>
<td>0.70</td>
<td>12.48</td>
<td>412</td>
<td>0.44</td>
<td>510</td>
<td>222.4</td>
</tr>
</tbody>
</table>

*values based on b.m.t.

---

**Figure A-5. Stress-strain curve for 0.90 mm thick G350 steel (coated)**

**Figure A-6. Stress-strain curve for 0.70 mm thick G350 steel (coated)**
A.3.2 Results of base panel shear tests and top plate buckling tests

Table A-6. Tensile coupon test results for 1.00 mm base panel material (G550)

<table>
<thead>
<tr>
<th>Coupon</th>
<th>Coating</th>
<th>$t_c$ (mm)</th>
<th>$t_b$ (mm)</th>
<th>$b$ (mm)</th>
<th>$f_y^*$ (MPa)</th>
<th>$\varepsilon_{0.2}^*$ (%)</th>
<th>$f_u^*$ (MPa)</th>
<th>$E^*$ (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>G550/1.00-1</td>
<td>coated</td>
<td>1.06</td>
<td>0.99</td>
<td>12.60</td>
<td>679</td>
<td>0.54</td>
<td>689</td>
<td>233.2</td>
</tr>
<tr>
<td>G550/1.00-2</td>
<td>coated</td>
<td>1.07</td>
<td>0.99</td>
<td>12.53</td>
<td>676</td>
<td>0.54</td>
<td>686</td>
<td>235.9</td>
</tr>
<tr>
<td>G550/1.00-3</td>
<td>coated</td>
<td>1.07</td>
<td>0.99</td>
<td>12.59</td>
<td>676</td>
<td>0.54</td>
<td>686</td>
<td>229.6</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td></td>
<td><strong>1.07</strong></td>
<td><strong>0.99</strong></td>
<td><strong>12.57</strong></td>
<td><strong>677</strong></td>
<td><strong>0.54</strong></td>
<td><strong>687</strong></td>
<td><strong>232.9</strong></td>
</tr>
<tr>
<td>G550/1.00-1</td>
<td>uncoated</td>
<td>n/a</td>
<td>0.99</td>
<td>12.62</td>
<td>656</td>
<td>0.55</td>
<td>667</td>
<td>225.7</td>
</tr>
<tr>
<td>G550/1.00-2</td>
<td>uncoated</td>
<td>n/a</td>
<td>1.00</td>
<td>12.57</td>
<td>663</td>
<td>0.55</td>
<td>668</td>
<td>223.8</td>
</tr>
<tr>
<td>G550/1.00-3</td>
<td>uncoated</td>
<td>n/a</td>
<td>0.99</td>
<td>12.52</td>
<td>663</td>
<td>0.55</td>
<td>673</td>
<td>223.0</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td></td>
<td>n/a</td>
<td>0.99</td>
<td>12.57</td>
<td>661</td>
<td>0.55</td>
<td>669</td>
<td>224.2</td>
</tr>
</tbody>
</table>

*values based on b.m.t.

![Stress-strain curve for 1.00 mm thick G550 steel (coated)](image)

Figure A-7. Stress-strain curve for 1.00 mm thick G550 steel (coated)

![Stress-strain curve for 1.00 mm thick G550 steel (uncoated)](image)

Figure A-8. Stress-strain curve for 1.00 mm thick G550 steel (uncoated)
Table A-7. Tensile coupon test results for 0.75 mm base panel material (G550)

<table>
<thead>
<tr>
<th>Coupon</th>
<th>Coating</th>
<th>$t_c$ (mm)</th>
<th>$t_b$ (mm)</th>
<th>$b$ (mm)</th>
<th>$f_y^*$ (MPa)</th>
<th>$\epsilon_{0.2}^*$ (%)</th>
<th>$f_u^*$ (MPa)</th>
<th>$E^*$ (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>G550/0.75-1</td>
<td>coated</td>
<td>0.81</td>
<td>0.74</td>
<td>12.59</td>
<td>696</td>
<td>0.54</td>
<td>699</td>
<td>236.5</td>
</tr>
<tr>
<td>G550/0.75-2</td>
<td>coated</td>
<td>0.81</td>
<td>0.74</td>
<td>12.57</td>
<td>695</td>
<td>0.54</td>
<td>699</td>
<td>236.3</td>
</tr>
<tr>
<td>G550/0.75-3</td>
<td>coated</td>
<td>0.81</td>
<td>0.75</td>
<td>12.55</td>
<td>700</td>
<td>0.55</td>
<td>702</td>
<td>232.5</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td></td>
<td><strong>0.81</strong></td>
<td><strong>0.74</strong></td>
<td><strong>12.57</strong></td>
<td><strong>697</strong></td>
<td><strong>0.54</strong></td>
<td><strong>700</strong></td>
<td><strong>235.1</strong></td>
</tr>
<tr>
<td>G550/0.75-1</td>
<td>uncoated</td>
<td>n/a</td>
<td>0.74</td>
<td>12.44</td>
<td>698</td>
<td>0.54</td>
<td>698</td>
<td>227.5</td>
</tr>
<tr>
<td>G550/0.75-2</td>
<td>uncoated</td>
<td>n/a</td>
<td>0.74</td>
<td>12.45</td>
<td>693</td>
<td>0.55</td>
<td>695</td>
<td>226.0</td>
</tr>
<tr>
<td>G550/0.75-3</td>
<td>uncoated</td>
<td>n/a</td>
<td>0.75</td>
<td>12.46</td>
<td>693</td>
<td>0.55</td>
<td>695</td>
<td>224.4</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td></td>
<td>n/a</td>
<td>0.74</td>
<td>12.45</td>
<td>695</td>
<td>0.55</td>
<td>696</td>
<td>226.0</td>
</tr>
</tbody>
</table>

* values based on b.m.t.

Figure A-9. Stress-strain curve for 0.75 mm thick G550 steel (coated)

Figure A-10. Stress-strain curve for 0.75 mm thick G550 steel (uncoated)
Appendix A. Tensile coupon tests

Table A-8. Tensile coupon test results for 0.90 mm web material (G350)

<table>
<thead>
<tr>
<th>Coupon</th>
<th>Coating</th>
<th>( t_c ) (mm)</th>
<th>( t_b ) (mm)</th>
<th>( b ) (mm)</th>
<th>( f_y^* ) (MPa)</th>
<th>( \varepsilon_{0.2}^* ) (%)</th>
<th>( f_u^* ) (MPa)</th>
<th>( E^* ) (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>G350/0.9-1</td>
<td>coated</td>
<td>0.94</td>
<td>0.89</td>
<td>12.61</td>
<td>391</td>
<td>0.43</td>
<td>486</td>
<td>221.4</td>
</tr>
<tr>
<td>G350/0.9-2</td>
<td>coated</td>
<td>0.95</td>
<td>0.89</td>
<td>12.60</td>
<td>392</td>
<td>0.44</td>
<td>490</td>
<td>222.9</td>
</tr>
<tr>
<td>G350/0.9-3</td>
<td>coated</td>
<td>0.94</td>
<td>0.89</td>
<td>12.60</td>
<td>390</td>
<td>0.43</td>
<td>486</td>
<td>227.7</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td>0.94</td>
<td>0.89</td>
<td>12.60</td>
<td>391</td>
<td>0.43</td>
<td>487</td>
<td>224.0</td>
</tr>
<tr>
<td>G350/0.9-1</td>
<td>uncoated</td>
<td>n/a</td>
<td>0.89</td>
<td>12.57</td>
<td>385</td>
<td>0.43</td>
<td>480</td>
<td>214.1</td>
</tr>
<tr>
<td>G350/0.9-2</td>
<td>uncoated</td>
<td>n/a</td>
<td>0.89</td>
<td>12.58</td>
<td>392</td>
<td>0.43</td>
<td>484</td>
<td>208.6</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td>n/a</td>
<td>0.89</td>
<td>12.57</td>
<td>389</td>
<td>0.43</td>
<td>482</td>
<td>211.4</td>
</tr>
</tbody>
</table>

* values based on b.m.t.

Figure A-11. Stress-strain curve for 0.90 mm thick G350 steel (coated)

Figure A-12. Stress-strain curve for 0.90 mm thick G350 steel (uncoated)
### Table A-9. Tensile coupon test results for 0.70 mm web material (G350)

<table>
<thead>
<tr>
<th>Coupon</th>
<th>Coating</th>
<th>$t_c$ (mm)</th>
<th>$t_b$ (mm)</th>
<th>$b$ (mm)</th>
<th>$f_y^*$ (MPa)</th>
<th>$\sigma_{0.2}^*$ (%)</th>
<th>$f_u^*$ (MPa)</th>
<th>$E^*$ (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>G350/0.7-1</td>
<td>coated</td>
<td>0.75</td>
<td>0.69</td>
<td>12.59</td>
<td>397</td>
<td>0.42</td>
<td>483</td>
<td>229.9</td>
</tr>
<tr>
<td>G350/0.7-2</td>
<td>coated</td>
<td>0.75</td>
<td>0.69</td>
<td>12.62</td>
<td>401</td>
<td>0.42</td>
<td>488</td>
<td>225.0</td>
</tr>
<tr>
<td>G350/0.7-3</td>
<td>coated</td>
<td>0.74</td>
<td>0.69</td>
<td>12.57</td>
<td>397</td>
<td>0.42</td>
<td>484</td>
<td>229.6</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td></td>
<td><strong>0.75</strong></td>
<td><strong>0.69</strong></td>
<td><strong>12.59</strong></td>
<td><strong>398</strong></td>
<td><strong>0.42</strong></td>
<td><strong>485</strong></td>
<td><strong>228.2</strong></td>
</tr>
<tr>
<td>G350/0.7-1</td>
<td>uncoated</td>
<td>n/a</td>
<td>0.69</td>
<td>12.53</td>
<td>381</td>
<td>0.43</td>
<td>469</td>
<td>210.1</td>
</tr>
<tr>
<td>G350/0.7-2</td>
<td>uncoated</td>
<td>n/a</td>
<td>0.69</td>
<td>12.54</td>
<td>381</td>
<td>0.44</td>
<td>468</td>
<td>208.4</td>
</tr>
<tr>
<td>G350/0.7-3</td>
<td>uncoated</td>
<td>n/a</td>
<td>0.69</td>
<td>12.56</td>
<td>385</td>
<td>0.44</td>
<td>471</td>
<td>202.2</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td></td>
<td>n/a</td>
<td>0.69</td>
<td>12.54</td>
<td>382</td>
<td>0.44</td>
<td>469</td>
<td>206.9</td>
</tr>
</tbody>
</table>

*values based on b.m.t.

### Figure A-13. Stress-strain curve for 0.70 mm thick G350 steel (coated)

### Figure A-14. Stress-strain curve for 0.70 mm thick G350 steel (uncoated)
Appendix A. Tensile coupon tests

Table A-10. Tensile coupon test results for top plate material (BLACKFORM)

<table>
<thead>
<tr>
<th>Coupon</th>
<th>Coating</th>
<th>tc (mm)</th>
<th>tb (mm)</th>
<th>ft* (MPa)</th>
<th>δ0.2 (%)</th>
<th>fu* (MPa)</th>
<th>E* (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BF/2.5-1</td>
<td>uncoated</td>
<td>2.52</td>
<td>12.32</td>
<td>299</td>
<td>0.39</td>
<td>417</td>
<td>224.9</td>
</tr>
<tr>
<td>BF/2.5-2</td>
<td>uncoated</td>
<td>2.52</td>
<td>12.37</td>
<td>295</td>
<td>0.39</td>
<td>406</td>
<td>218.3</td>
</tr>
<tr>
<td>BF/2.5-3</td>
<td>uncoated</td>
<td>2.52</td>
<td>12.30</td>
<td>290</td>
<td>0.40</td>
<td>411</td>
<td>212.4</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td>2.52</td>
<td>12.33</td>
<td>295</td>
<td>0.39</td>
<td>411</td>
<td>218.5</td>
</tr>
<tr>
<td>BF/1.6-1</td>
<td>uncoated</td>
<td>1.56</td>
<td>12.34</td>
<td>309</td>
<td>0.39</td>
<td>385</td>
<td>223.4</td>
</tr>
<tr>
<td>BF/1.6-2</td>
<td>uncoated</td>
<td>1.56</td>
<td>12.34</td>
<td>307</td>
<td>0.39</td>
<td>386</td>
<td>234.3</td>
</tr>
<tr>
<td>BF/1.6-3</td>
<td>uncoated</td>
<td>1.56</td>
<td>12.34</td>
<td>313</td>
<td>0.39</td>
<td>386</td>
<td>214.6</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td>1.56</td>
<td>12.34</td>
<td>310</td>
<td>0.39</td>
<td>386</td>
<td>224.1</td>
</tr>
</tbody>
</table>

* values based on b.m.t.

Figure A-15. Stress-strain curve for 2.50 mm thick BLACKFORM steel (uncoated)

Figure A-16. Stress-strain curve for 1.60 mm thick BLACKFORM steel (uncoated)
Appendix A. Tensile coupon tests

Table A-11. Tensile coupon test results for top plate material (G250)

<table>
<thead>
<tr>
<th>Coupon</th>
<th>Coating</th>
<th>thickness ( t_c ) (mm)</th>
<th>thickness ( t_b ) (mm)</th>
<th>width ( b ) (mm)</th>
<th>( f_y^* ) (MPa)</th>
<th>( \varepsilon_{0.2}^* ) (%)</th>
<th>( f_u^* ) (MPa)</th>
<th>( E^* ) (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HA250/1.85-1</td>
<td>uncoated</td>
<td>1.83</td>
<td>12.30</td>
<td>258</td>
<td>0.38</td>
<td>354</td>
<td>222.2</td>
<td></td>
</tr>
<tr>
<td>HA250/1.85-2</td>
<td>uncoated</td>
<td>1.84</td>
<td>12.30</td>
<td>264</td>
<td>0.37</td>
<td>355</td>
<td>235.6</td>
<td></td>
</tr>
<tr>
<td>HA250/1.85-3</td>
<td>uncoated</td>
<td>1.84</td>
<td>12.31</td>
<td>262</td>
<td>0.37</td>
<td>353</td>
<td>218.0</td>
<td></td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td></td>
<td>1.84</td>
<td>12.30</td>
<td>261</td>
<td>0.37</td>
<td>354</td>
<td>225.3</td>
<td></td>
</tr>
</tbody>
</table>

* values based on b.m.t.

Table A-12. Tensile coupon test results for top plate material (HA70T)

<table>
<thead>
<tr>
<th>Coupon</th>
<th>Coating</th>
<th>thickness ( t_c ) (mm)</th>
<th>thickness ( t_b ) (mm)</th>
<th>width ( b ) (mm)</th>
<th>( f_y^* ) (MPa)</th>
<th>( \varepsilon_{0.2}^* ) (%)</th>
<th>( f_u^* ) (MPa)</th>
<th>( E^* ) (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HA70T/1.9-1</td>
<td>uncoated</td>
<td>1.90</td>
<td>12.32</td>
<td>489</td>
<td>0.48</td>
<td>498</td>
<td>211.6</td>
<td></td>
</tr>
<tr>
<td>HA70T/1.9-2</td>
<td>uncoated</td>
<td>1.90</td>
<td>12.31</td>
<td>485</td>
<td>0.49</td>
<td>496</td>
<td>202.3</td>
<td></td>
</tr>
<tr>
<td>HA70T/1.9-3</td>
<td>uncoated</td>
<td>1.90</td>
<td>12.31</td>
<td>490</td>
<td>0.49</td>
<td>500</td>
<td>198.0</td>
<td></td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td></td>
<td>1.90</td>
<td>12.31</td>
<td>488</td>
<td>0.49</td>
<td>498</td>
<td>204.0</td>
<td></td>
</tr>
</tbody>
</table>

* values based on b.m.t.

Figure A-17. Stress-strain curve for 1.85 mm thick HA250 steel (uncoated)

Figure A-18. Stress-strain curve for 1.90 mm thick HA70T steel (uncoated)
Appendix A. Tensile coupon tests

A.4 TENSILE COUPON TESTS OF TOX SHEAR TEST MATERIAL

Tensile testing of coupons was carried out to determine the mechanical properties of the material used in the TOX shear test (Chapter 3.1). The milled coupons had a nominal width of 12.5 mm at the central portion of the constant gauge length and 80 mm of parallel length. The transition radius $r$ of the 200 mm long coupon was 16 mm. All materials were tested in the longitudinal, i.e. rolling, direction and mechanical properties were based on the base metal thickness (b.m.t.) although the coupons were tested in their original coated state. Tests were undertaken in a displacement controlled INSTRON universal testing machine (Model No. 6027) with strains measured by two standard 50 mm gauge-length INSTRON clip-on extensometers (Model No. 2630-100) placed on each face of the coupon. Stress-strain curves were obtained from the recordings of load and strains (average value of both extensometers) and are presented in Figure A-19 to Figure A-24.

### Table A-13. Tensile coupon test results for punch side material (G550 and HA70T)

<table>
<thead>
<tr>
<th>Coupon</th>
<th>Coating</th>
<th>$t_c$ (mm)</th>
<th>$t_b$ (mm)</th>
<th>$b$ (mm)</th>
<th>$f_y^*$ (MPa)</th>
<th>$\sigma_{0.2}^*$ (%)</th>
<th>$f_u^*$ (MPa)</th>
<th>$E^*$ (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>G550/1.00-1</td>
<td>coated</td>
<td>1.07</td>
<td>0.97</td>
<td>13.34</td>
<td>715</td>
<td>0.54</td>
<td>715</td>
<td>230.6</td>
</tr>
<tr>
<td>G550/1.00-2</td>
<td>coated</td>
<td>1.07</td>
<td>0.97</td>
<td>13.06</td>
<td>722</td>
<td>0.51</td>
<td>726</td>
<td>227.5</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td></td>
<td><strong>1.07</strong></td>
<td><strong>0.97</strong></td>
<td><strong>13.20</strong></td>
<td><strong>719</strong></td>
<td><strong>0.53</strong></td>
<td><strong>721</strong></td>
<td><strong>229.1</strong></td>
</tr>
<tr>
<td>G550/0.75-1</td>
<td>coated</td>
<td>0.81</td>
<td>0.73</td>
<td>13.09</td>
<td>649</td>
<td>0.53</td>
<td>658</td>
<td>230.5</td>
</tr>
<tr>
<td>G550/0.75-2</td>
<td>coated</td>
<td>0.83</td>
<td>0.75</td>
<td>13.35</td>
<td>695</td>
<td>0.54</td>
<td>698</td>
<td>227.9</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td></td>
<td><strong>0.82</strong></td>
<td><strong>0.74</strong></td>
<td><strong>13.22</strong></td>
<td><strong>672</strong></td>
<td><strong>0.54</strong></td>
<td><strong>678</strong></td>
<td><strong>229.2</strong></td>
</tr>
<tr>
<td>ZHT G550/0.55-1</td>
<td>coated</td>
<td>0.63</td>
<td>0.53</td>
<td>13.37</td>
<td>n/a</td>
<td>n/a</td>
<td>764</td>
<td>237.5</td>
</tr>
<tr>
<td>ZHT G550/0.55-2</td>
<td>coated</td>
<td>0.63</td>
<td>0.54</td>
<td>13.37</td>
<td>n/a</td>
<td>n/a</td>
<td>777</td>
<td>236.9</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td></td>
<td><strong>0.63</strong></td>
<td><strong>0.53</strong></td>
<td><strong>13.37</strong></td>
<td>n/a</td>
<td>n/a</td>
<td><strong>771</strong></td>
<td><strong>237.2</strong></td>
</tr>
<tr>
<td>HA70T/1.60-1</td>
<td>uncoated</td>
<td>n/a</td>
<td>1.62</td>
<td>13.37</td>
<td>407</td>
<td>0.44</td>
<td>503</td>
<td>213.5</td>
</tr>
<tr>
<td>HA70T/1.60-2</td>
<td>uncoated</td>
<td>n/a</td>
<td>1.62</td>
<td>13.32</td>
<td>404</td>
<td>0.47</td>
<td>495</td>
<td>205.5</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td></td>
<td><strong>n/a</strong></td>
<td><strong>1.62</strong></td>
<td><strong>13.35</strong></td>
<td><strong>405</strong></td>
<td><strong>0.46</strong></td>
<td><strong>499</strong></td>
<td><strong>209.5</strong></td>
</tr>
</tbody>
</table>

* values based on b.m.t.

### Table A-14. Tensile coupon test results for die side material (G300)

<table>
<thead>
<tr>
<th>Coupon</th>
<th>Coating</th>
<th>$t_c$ (mm)</th>
<th>$t_b$ (mm)</th>
<th>$b$ (mm)</th>
<th>$f_y^*$ (MPa)</th>
<th>$\sigma_{0.2}^*$ (%)</th>
<th>$f_u^*$ (MPa)</th>
<th>$E^*$ (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>G300/0.90-1</td>
<td>coated</td>
<td>0.94</td>
<td>0.88</td>
<td>13.06</td>
<td>330</td>
<td>0.38</td>
<td>422</td>
<td>235.1</td>
</tr>
<tr>
<td>G300/0.90-2</td>
<td>coated</td>
<td>0.94</td>
<td>0.88</td>
<td>13.38</td>
<td>323</td>
<td>0.39</td>
<td>416</td>
<td>236.2</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td></td>
<td><strong>0.94</strong></td>
<td><strong>0.88</strong></td>
<td><strong>13.22</strong></td>
<td><strong>327</strong></td>
<td><strong>0.39</strong></td>
<td><strong>419</strong></td>
<td><strong>235.7</strong></td>
</tr>
<tr>
<td>G300/0.70-1</td>
<td>coated</td>
<td>0.82</td>
<td>0.70</td>
<td>13.18</td>
<td>342</td>
<td>0.41</td>
<td>416</td>
<td>237.5</td>
</tr>
<tr>
<td>G300/0.70-2</td>
<td>coated</td>
<td>0.82</td>
<td>0.70</td>
<td>13.16</td>
<td>343</td>
<td>0.42</td>
<td>419</td>
<td>235.3</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td></td>
<td><strong>0.82</strong></td>
<td><strong>0.70</strong></td>
<td><strong>13.17</strong></td>
<td><strong>343</strong></td>
<td><strong>0.42</strong></td>
<td><strong>418</strong></td>
<td><strong>236.4</strong></td>
</tr>
</tbody>
</table>

* values based on b.m.t.
Appendix A. Tensile coupon tests

Figure A-19. Stress-strain curve for 1.00 mm thick G550 steel (coated)

Figure A-20. Stress-strain curve for 0.75 mm thick G550 steel (coated)

Figure A-21. Stress-strain curve for 0.55 mm thick ZHT G550 steel (coated)
Appendix A. Tensile coupon tests

Figure A-22. Stress-strain curve for 1.60 mm thick HA70T steel (uncoated)

Figure A-23. Stress-strain curve for 0.90 mm thick G300 steel (coated)

Figure A-24. Stress-strain curve for 0.70 mm thick G300 steel (coated)
A.5 REFERENCES


Maladakis, K., and Ayoub, G., (1994), "Investigation on Ductility Requirements of High Strength Steels with Perforations", *Bachelor of Engineering Honours Thesis*, School of Civil and Mining Engineering, University of Sydney, Sydney, Australia.


APPENDIX B
TEST RESULTS FOR TOX SHEAR TESTS

Results for each of the 42 single lap shear tests, carried out to better understand the shear behaviour of round TOX press-joints when dissimilar material combinations in regards to thickness and steel grades are used for the hybrid steel deck, are presented in this Appendix. Descriptions of the various test set-ups as well as a discussion of the test results are given in Chapter 3.1.

Two standard 50 mm gauge-length extensometers were attached directly to the specimen at the overlapping region to measure the deformation of the TOX connector. In the cases where the extensometers could not be used (e.g. for three connectors in a row) or fell off during testing, crosshead movement is presented instead of slip. For the back-to-back tests, two specimens were tested simultaneously and indicated with Part A and Part B, respectively. Whenever both extensometers were attached to one specimen only, the average of both slip measurements was taken.

Each specimen was given a distinct alpha-numeric code, determined from basic information of the specimen. The initial letter represents the test set-up (A for the back-to-back, and B for the modified test set-up), followed by two numbers which state the amount of connectors in a row and their nominal outside diameter in millimetres. The following three numbers represent the nominal thickness of the punch side material, followed by two numbers which represent the nominal thickness of the die side material. Both thicknesses are given in millimetres times 100. The final number, separated by a dash, states the specimen number. Examples of the code are given below.

<table>
<thead>
<tr>
<th>Test set-up</th>
<th>No. of connectors</th>
<th>Outside diameter</th>
<th>Thickness punch side</th>
<th>Thickness die side</th>
<th>Specimen number</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>3</td>
<td>6</td>
<td>055</td>
<td>70</td>
<td>2</td>
</tr>
<tr>
<td>B</td>
<td>1</td>
<td>8</td>
<td>100</td>
<td>90</td>
<td>1</td>
</tr>
</tbody>
</table>
Appendix B. Tests results for TOX shear tests

First test series: back-to-back test set-up

Figure B-1. Specimen A1605570-1

Figure B-2. Specimen A1605570-2

Figure B-3. Specimen A1605570-3
Appendix B. Tests results for TOX shear tests

Figure B-4. Specimen A1605590-1

Figure B-5. Specimen A1605590-2

Figure B-6. Specimen A1605590-3
Appendix B. Tests results for TOX shear tests

Figure B-7. Specimen A1805570-1

Figure B-8. Specimen A1805570-2

Figure B-9. Specimen A1805570-3
Appendix B. Tests results for TOX shear tests

Figure B-10. Specimen A1805590-1

Figure B-11. Specimen A1805590-2
Appendix B. Tests results for TOX shear tests

First test series: back-to-back test set-up (three connectors in a row)

Figure B-12. Specimen A3605570-1

Figure B-13. Specimen A3605570-2

Figure B-14. Specimen A3605570-3
Appendix B. Tests results for TOX shear tests

Figure B-15. Specimen A3605590-1

Figure B-16. Specimen A3605590-2

Figure B-17. Specimen A3605590-3
Appendix B. Tests results for TOX shear tests

Figure B-18. Specimen A3805570-1

Figure B-19. Specimen A3805570-2

Figure B-20. Specimen A3805570-3
Appendix B. Tests results for TOX shear tests

First test series: back-to-back test set-up (1.60 mm thick punch side material)

Figure B-21. Specimen A1816090-1

Figure B-22. Specimen A1816090-2

Figure B-23. Specimen A1816090-3
Appendix B. Tests results for TOX shear tests

Figure B-24. Specimen A1816070-1

Figure B-25. Specimen A1816070-2

Figure B-26. Specimen A1816070-3
Appendix B. Tests results for TOX shear tests

Second test series: back-to-back test set-up

Figure B-27. Specimen A1805570

Figure B-28. Specimen A1807570

Figure B-29. Specimen A1810070
Appendix B. Tests results for TOX shear tests

Second test series: modified test set-up

Figure B-30. Specimen B05590-1

Figure B-31. Specimen B05590-2
Appendix B. Tests results for TOX shear tests

Figure B-32. Specimen B05570-1

Figure B-33. Specimen B05570-2
Appendix B. Tests results for TOX shear tests

Figure B-34. Specimen B07590-1

Figure B-35. Specimen B07570-1

Figure B-36. Specimen B07570-2
Appendix B. Tests results for TOX shear tests

Figure B-37. Specimen B10090-1

Figure B-38. Specimen B10090-2

Figure B-39. Specimen B10090-3
Appendix B. Tests results for TOX shear tests

Figure B-40. Specimen B10070-1

Figure B-41. Specimen B10070-2

Figure B-42. Specimen B10070-3
APPENDIX C
TEST RESULTS FOR TOX INFLUENCE ON STEEL PERFORMANCE

Results for each of the 58 tensile coupons (48 incorporating round TOX press-joints) tested to
investigate the influence of the clinch connectors on the performance of sheet steel, in particular of
high-strength steel (G550), are presented in this Appendix. A description of the test set-up as well as a
discussion of the test results is given in Chapter 3.2.

The geometry of all test specimens is listed in Table C-1 to Table C-4, where \( t_c \) and \( t_b \) are the
coated and base metal thickness, respectively, and \( b \) the width of the tensile coupon. The inside
diameter \( i \) of the press-joint caused by the imprint of the punch is also given. Based on the base metal
thickness \( t_b \) the gross cross-sectional area \( A_{gb} \) (assuming the press-joint does not reduce the cross-
section at all) and the net cross-sectional area \( A_{nb} \) (assuming the press-joint reduces the cross-section
as much as an actual hole) are calculated. Similarly, the values for \( A_{sc} \) and \( A_{hc} \) were calculated using
the coated metal thickness \( t_c \). Since the extruded material of the press-joint can still carry some of the
stress, it would be overly conservative to treat the press-joint as a hole of the same inside diameter \( i \)
as the joint itself, but it could be considered as a hole with a reduced diameter \( i_{red} \). The calculated values
for \( i_{red} \) are also listed in the tables.

Each specimen was given a distinct alpha-numeric code, e.g. 550(100)30A-2, determined from
basic information. The initial three numbers characterize the type of material used for the tensile
coupon, e.g. G550, followed by a further three numbers in brackets which represent its nominal base
metal thickness in millimetres times 100, e.g. 1.00 mm. The constant gauge width of the coupon is
stated in the subsequent two numbers, e.g. 30 mm, followed by a letter indicating the specimen type,
i.e. solid (S), clinching direction (A) or clinching direction (B). The specimen number, e.g. 2 is
separated by a dash.
### Appendix C. Tests results for TOX influence on steel performance

#### Table C-1. Geometry G300, 0.55 mm

<table>
<thead>
<tr>
<th>Code</th>
<th>tc</th>
<th>tb</th>
<th>b</th>
<th>i</th>
<th>hช</th>
<th>Aχс</th>
<th>Aχθ</th>
<th>Aχп</th>
<th>Aχρ</th>
</tr>
</thead>
<tbody>
<tr>
<td>300(055)S-1</td>
<td>0.61</td>
<td>0.55</td>
<td>20.12</td>
<td>n/a</td>
<td>n/a</td>
<td>12.27</td>
<td>n/a</td>
<td>11.07</td>
<td>n/a</td>
</tr>
<tr>
<td>300(055)S-2</td>
<td>0.60</td>
<td>0.55</td>
<td>20.08</td>
<td>n/a</td>
<td>n/a</td>
<td>12.05</td>
<td>n/a</td>
<td>11.04</td>
<td>n/a</td>
</tr>
<tr>
<td>300(055)S-3</td>
<td>0.60</td>
<td>0.55</td>
<td>20.05</td>
<td>n/a</td>
<td>n/a</td>
<td>12.03</td>
<td>n/a</td>
<td>11.03</td>
<td>n/a</td>
</tr>
<tr>
<td>300(055)20A-1</td>
<td>0.60</td>
<td>0.55</td>
<td>20.10</td>
<td>7.5</td>
<td>0.74</td>
<td>12.06</td>
<td>7.77</td>
<td>11.06</td>
<td>7.12</td>
</tr>
<tr>
<td>300(055)20A-2</td>
<td>0.60</td>
<td>0.55</td>
<td>20.10</td>
<td>7.5</td>
<td>0.74</td>
<td>12.06</td>
<td>7.77</td>
<td>11.06</td>
<td>7.12</td>
</tr>
<tr>
<td>300(055)30A-1</td>
<td>0.61</td>
<td>0.55</td>
<td>30.00</td>
<td>7.5</td>
<td>0.50</td>
<td>18.30</td>
<td>13.94</td>
<td>16.50</td>
<td>12.57</td>
</tr>
<tr>
<td>300(055)30A-2</td>
<td>0.60</td>
<td>0.55</td>
<td>30.10</td>
<td>7.5</td>
<td>0.60</td>
<td>18.06</td>
<td>13.77</td>
<td>16.56</td>
<td>12.62</td>
</tr>
<tr>
<td>300(055)40A-1</td>
<td>0.60</td>
<td>0.55</td>
<td>40.03</td>
<td>7.5</td>
<td>-0.01</td>
<td>24.02</td>
<td>19.73</td>
<td>22.02</td>
<td>18.08</td>
</tr>
<tr>
<td>300(055)40A-2</td>
<td>0.60</td>
<td>0.55</td>
<td>40.16</td>
<td>7.5</td>
<td>0.67</td>
<td>24.09</td>
<td>19.80</td>
<td>22.08</td>
<td>18.15</td>
</tr>
</tbody>
</table>

#### Table C-2. Geometry G300, 0.85 mm

<table>
<thead>
<tr>
<th>Code</th>
<th>tc</th>
<th>tb</th>
<th>b</th>
<th>i</th>
<th>hช</th>
<th>Aχс</th>
<th>Aχθ</th>
<th>Aχп</th>
<th>Aχρ</th>
</tr>
</thead>
<tbody>
<tr>
<td>300(085)S-1</td>
<td>0.92</td>
<td>0.86</td>
<td>20.10</td>
<td>n/a</td>
<td>n/a</td>
<td>18.49</td>
<td>n/a</td>
<td>17.29</td>
<td>n/a</td>
</tr>
<tr>
<td>300(085)S-2</td>
<td>0.92</td>
<td>0.86</td>
<td>20.03</td>
<td>n/a</td>
<td>n/a</td>
<td>18.23</td>
<td>n/a</td>
<td>17.23</td>
<td>n/a</td>
</tr>
<tr>
<td>300(085)20A-1</td>
<td>0.91</td>
<td>0.86</td>
<td>19.93</td>
<td>6.82</td>
<td>0.50</td>
<td>18.14</td>
<td>11.93</td>
<td>17.14</td>
<td>11.27</td>
</tr>
<tr>
<td>300(085)20A-2</td>
<td>0.91</td>
<td>0.86</td>
<td>19.95</td>
<td>6.82</td>
<td>0.57</td>
<td>18.15</td>
<td>11.95</td>
<td>17.16</td>
<td>11.29</td>
</tr>
<tr>
<td>300(085)30A-1</td>
<td>0.90</td>
<td>0.85</td>
<td>29.95</td>
<td>6.82</td>
<td>0.11</td>
<td>26.96</td>
<td>20.82</td>
<td>25.46</td>
<td>19.66</td>
</tr>
<tr>
<td>300(085)30A-2</td>
<td>0.90</td>
<td>0.85</td>
<td>29.94</td>
<td>6.82</td>
<td>-0.02</td>
<td>26.95</td>
<td>20.81</td>
<td>25.45</td>
<td>19.65</td>
</tr>
<tr>
<td>300(085)40A-1</td>
<td>0.91</td>
<td>0.86</td>
<td>39.97</td>
<td>6.82</td>
<td>0.51</td>
<td>36.37</td>
<td>30.17</td>
<td>34.37</td>
<td>28.51</td>
</tr>
<tr>
<td>300(085)40A-2</td>
<td>0.92</td>
<td>0.86</td>
<td>40.00</td>
<td>6.82</td>
<td>-0.27</td>
<td>36.80</td>
<td>30.53</td>
<td>34.40</td>
<td>28.53</td>
</tr>
</tbody>
</table>

#### Table C-3. Geometry G550, 0.75 mm

<table>
<thead>
<tr>
<th>Code</th>
<th>tc</th>
<th>tb</th>
<th>b</th>
<th>i</th>
<th>hช</th>
<th>Aχс</th>
<th>Aχθ</th>
<th>Aχп</th>
<th>Aχρ</th>
</tr>
</thead>
<tbody>
<tr>
<td>550(075)S-1</td>
<td>0.84</td>
<td>0.76</td>
<td>20.10</td>
<td>n/a</td>
<td>n/a</td>
<td>16.88</td>
<td>n/a</td>
<td>15.28</td>
<td>n/a</td>
</tr>
<tr>
<td>550(075)S-2</td>
<td>0.84</td>
<td>0.76</td>
<td>20.10</td>
<td>n/a</td>
<td>n/a</td>
<td>16.88</td>
<td>n/a</td>
<td>15.28</td>
<td>n/a</td>
</tr>
<tr>
<td>550(075)20A-1</td>
<td>0.84</td>
<td>0.76</td>
<td>20.20</td>
<td>6.83</td>
<td>4.17</td>
<td>16.97</td>
<td>11.23</td>
<td>15.35</td>
<td>10.16</td>
</tr>
<tr>
<td>550(075)20A-2</td>
<td>0.84</td>
<td>0.76</td>
<td>20.15</td>
<td>6.83</td>
<td>4.53</td>
<td>16.93</td>
<td>11.19</td>
<td>15.31</td>
<td>10.12</td>
</tr>
<tr>
<td>550(075)30A-1</td>
<td>0.84</td>
<td>0.76</td>
<td>30.00</td>
<td>6.83</td>
<td>4.53</td>
<td>25.20</td>
<td>19.46</td>
<td>22.80</td>
<td>17.61</td>
</tr>
<tr>
<td>550(075)30A-2</td>
<td>0.85</td>
<td>0.76</td>
<td>30.00</td>
<td>6.83</td>
<td>4.57</td>
<td>25.50</td>
<td>19.69</td>
<td>22.80</td>
<td>17.61</td>
</tr>
<tr>
<td>550(075)40A-1</td>
<td>0.84</td>
<td>0.76</td>
<td>40.20</td>
<td>6.83</td>
<td>4.79</td>
<td>33.77</td>
<td>28.03</td>
<td>30.55</td>
<td>25.36</td>
</tr>
<tr>
<td>550(075)40A-2</td>
<td>0.85</td>
<td>0.76</td>
<td>40.10</td>
<td>6.83</td>
<td>4.53</td>
<td>34.09</td>
<td>28.28</td>
<td>30.48</td>
<td>25.29</td>
</tr>
</tbody>
</table>

**Note:** The values in the table represent specific measurements for each code, where `tc` and `tb` are thickness measurements, `b` is a length measurement, `i` and `hช` are other specific measurements related to the test results, and `Aχс`, `Aχθ`, `Aχп`, and `Aχρ` represent areas or other relevant metrics for the tests conducted.
Appendix C. Tests results for TOX influence on steel performance

Table C-4. Geometry G550, 1.00 mm

<table>
<thead>
<tr>
<th>Code</th>
<th>$t_c$</th>
<th>$t_b$</th>
<th>$b$</th>
<th>$i$</th>
<th>$A_{hc}$</th>
<th>$A_{sc}$</th>
<th>$A_{sb}$</th>
<th>$A_{hb}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(mm)</td>
<td>(mm)</td>
<td>(mm)</td>
<td>(mm)</td>
<td>(mm$^2$)</td>
<td>(mm$^2$)</td>
<td>(mm$^2$)</td>
<td>(mm$^2$)</td>
</tr>
<tr>
<td>550(100)S-1</td>
<td>1.05</td>
<td>0.99</td>
<td>20.18</td>
<td>n/a</td>
<td>21.19</td>
<td>n/a</td>
<td>19.78</td>
<td>n/a</td>
</tr>
<tr>
<td>550(100)S-2</td>
<td>1.06</td>
<td>0.99</td>
<td>20.03</td>
<td>n/a</td>
<td>21.23</td>
<td>n/a</td>
<td>19.83</td>
<td>n/a</td>
</tr>
<tr>
<td>550(100)S-3</td>
<td>1.06</td>
<td>0.99</td>
<td>20.04</td>
<td>n/a</td>
<td>21.24</td>
<td>n/a</td>
<td>19.84</td>
<td>n/a</td>
</tr>
<tr>
<td>550(100)20A-1</td>
<td>1.06</td>
<td>0.99</td>
<td>20.15</td>
<td>6.70</td>
<td>3.19</td>
<td>21.36</td>
<td>14.26</td>
<td>19.95</td>
</tr>
<tr>
<td>550(100)20A-2</td>
<td>1.06</td>
<td>0.99</td>
<td>20.12</td>
<td>6.70</td>
<td>3.27</td>
<td>21.33</td>
<td>14.23</td>
<td>19.92</td>
</tr>
<tr>
<td>550(100)30A-1</td>
<td>1.07</td>
<td>0.99</td>
<td>29.75</td>
<td>6.70</td>
<td>2.09</td>
<td>31.83</td>
<td>24.66</td>
<td>29.45</td>
</tr>
<tr>
<td>550(100)30A-2</td>
<td>1.06</td>
<td>0.99</td>
<td>29.90</td>
<td>6.70</td>
<td>2.74</td>
<td>31.69</td>
<td>24.59</td>
<td>29.60</td>
</tr>
<tr>
<td>550(100)40A-1</td>
<td>1.07</td>
<td>0.99</td>
<td>40.12</td>
<td>6.70</td>
<td>1.15</td>
<td>42.93</td>
<td>35.76</td>
<td>39.72</td>
</tr>
<tr>
<td>550(100)40A-2</td>
<td>1.06</td>
<td>0.99</td>
<td>40.10</td>
<td>6.70</td>
<td>2.33</td>
<td>42.51</td>
<td>35.40</td>
<td>39.70</td>
</tr>
<tr>
<td>550(100)20B-1</td>
<td>1.06</td>
<td>0.99</td>
<td>20.15</td>
<td>6.20</td>
<td>2.32</td>
<td>21.36</td>
<td>14.79</td>
<td>19.95</td>
</tr>
<tr>
<td>550(100)20B-2</td>
<td>1.06</td>
<td>0.99</td>
<td>20.17</td>
<td>6.20</td>
<td>2.29</td>
<td>21.38</td>
<td>14.81</td>
<td>19.97</td>
</tr>
<tr>
<td>550(100)30B-1</td>
<td>1.07</td>
<td>0.99</td>
<td>30.05</td>
<td>6.20</td>
<td>1.11</td>
<td>32.15</td>
<td>25.52</td>
<td>29.75</td>
</tr>
<tr>
<td>550(100)30B-2</td>
<td>1.07</td>
<td>0.99</td>
<td>29.75</td>
<td>6.20</td>
<td>1.90</td>
<td>31.83</td>
<td>25.20</td>
<td>29.45</td>
</tr>
<tr>
<td>550(100)40B-1</td>
<td>1.06</td>
<td>0.99</td>
<td>40.15</td>
<td>6.20</td>
<td>1.81</td>
<td>42.56</td>
<td>35.99</td>
<td>39.75</td>
</tr>
<tr>
<td>550(100)40B-2</td>
<td>1.06</td>
<td>0.99</td>
<td>40.20</td>
<td>6.20</td>
<td>1.57</td>
<td>42.61</td>
<td>36.04</td>
<td>39.80</td>
</tr>
</tbody>
</table>

The basic material properties, i.e. yield stress ($f_y$), ultimate strength ($f_u$) and Young’s modulus ($E$) of the sheet steels used in the test series, were obtained through tensile testing of the coated solid specimens, viz. specimens without a press-joint. The results, including the geometry and the strain at 0.2 % proof stress $\varepsilon_{0.2}$, are presented in Table C-5 for the G300 material and in Table C-6 for the G550 material and were based on the base metal thickness (b.m.t.) of the test specimen. The base metal thicknesses were obtained by acid-etching coated material in a diluted hydrochloric acid bath (four parts acid and one part distilled water). The stress-strain curves of the various materials are given in Figure C-11 to Figure C-14.
Appendix C. Tests results for TOX influence on steel performance

Figure C-1. Load-displacement curves for 0.85 mm thick G300 steel (Version A)

Figure C-2. Load-displacement curves for 0.85 mm thick G300 steel (Version B)
Appendix C. Tests results for TOX influence on steel performance

Figure C-3. Load-displacement curves for 0.55 mm thick G300 steel (Version A)

Figure C-4. Load-displacement curves for 0.55 mm thick G300 steel (Version B)
Appendix C. Tests results for TOX influence on steel performance

Figure C-5. Load-displacement curves for 1.00 mm thick G550 steel (Version A)

Figure C-6. Load-displacement curves for 1.00 mm thick G550 steel (Version B)
Appendix C. Tests results for TOX influence on steel performance

Figure C-7. Load-displacement curves for 0.75 mm thick G550 steel (Version A)

Figure C-8. Load-displacement curves for 0.75 mm thick G550 steel (Version B)
Appendix C. Tests results for TOX influence on steel performance

Figure C-9. Load-displacement curves for solid G300 steel

Figure C-10. Load-displacement curves for solid G550 steel
### Table C-5. Tensile coupon test results (G300)

<table>
<thead>
<tr>
<th>Coupon</th>
<th>Coating</th>
<th>$t_c$ (mm)</th>
<th>$t_b$ (mm)</th>
<th>$b$ (mm)</th>
<th>$f_y^*$ (MPa)</th>
<th>$\sigma_{0.2}^*$ (%)</th>
<th>$f_u^*$ (MPa)</th>
<th>$E^*$ (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>300(085)S-1 coated</td>
<td>0.92</td>
<td>0.86</td>
<td>20.10</td>
<td>331.4</td>
<td>0.36</td>
<td>379.3</td>
<td>226.6</td>
<td></td>
</tr>
<tr>
<td>300(085)S-2 coated</td>
<td>0.91</td>
<td>0.86</td>
<td>20.03</td>
<td>329.7</td>
<td>0.38</td>
<td>377.3</td>
<td>226.0</td>
<td></td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>0.92</strong></td>
<td><strong>0.86</strong></td>
<td><strong>20.07</strong></td>
<td><strong>330.6</strong></td>
<td><strong>0.37</strong></td>
<td><strong>378.3</strong></td>
<td><strong>226.3</strong></td>
<td></td>
</tr>
<tr>
<td>300(055)S-1 coated</td>
<td>0.61</td>
<td>0.55</td>
<td>20.12</td>
<td>361.0</td>
<td>0.39</td>
<td>395.0</td>
<td>217.7</td>
<td></td>
</tr>
<tr>
<td>300(055)S-2 coated</td>
<td>0.60</td>
<td>0.55</td>
<td>20.08</td>
<td>365.8</td>
<td>0.39</td>
<td>398.2</td>
<td>226.9</td>
<td></td>
</tr>
<tr>
<td>300(055)S-3 coated</td>
<td>0.60</td>
<td>0.55</td>
<td>20.05</td>
<td>370.3</td>
<td>0.39</td>
<td>398.8</td>
<td>220.6</td>
<td></td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>0.60</strong></td>
<td><strong>0.55</strong></td>
<td><strong>20.08</strong></td>
<td><strong>365.4</strong></td>
<td><strong>0.39</strong></td>
<td><strong>397.4</strong></td>
<td><strong>221.7</strong></td>
<td></td>
</tr>
</tbody>
</table>

*values based on b.m.t.

---

![Figure C-11. Stress-strain curve for 0.85 mm thick G300 steel (coated)](image1)

![Figure C-12. Stress-strain curve for 0.55 mm thick G300 steel (coated)](image2)
Table C-6. Tensile coupon test results (G550)

<table>
<thead>
<tr>
<th>Coupon</th>
<th>Coating</th>
<th>t_0 (mm)</th>
<th>t_b (mm)</th>
<th>b (mm)</th>
<th>f_y* (MPa)</th>
<th>a_{0.2}* (%)</th>
<th>f_u* (MPa)</th>
<th>E* (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>550(100)S-1</td>
<td>coated</td>
<td>1.05</td>
<td>0.98</td>
<td>20.18</td>
<td>609.3</td>
<td>0.51</td>
<td>622.5</td>
<td>221.6</td>
</tr>
<tr>
<td>550(100)S-2</td>
<td>coated</td>
<td>1.06</td>
<td>0.99</td>
<td>20.03</td>
<td>629.6</td>
<td>0.51</td>
<td>638.7</td>
<td>223.8</td>
</tr>
<tr>
<td>550(100)S-3</td>
<td>coated</td>
<td>1.06</td>
<td>0.99</td>
<td>20.04</td>
<td>634.3</td>
<td>0.52</td>
<td>645.8</td>
<td>222.7</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td>1.06</td>
<td>0.99</td>
<td>20.10</td>
<td>624.4</td>
<td>0.51</td>
<td>635.7</td>
<td>222.7</td>
</tr>
<tr>
<td>550(075)S-1</td>
<td>coated</td>
<td>0.84</td>
<td>0.76</td>
<td>20.10</td>
<td>714.0</td>
<td>0.52</td>
<td>716.7</td>
<td>228.5</td>
</tr>
<tr>
<td>550(075)S-2</td>
<td>coated</td>
<td>0.84</td>
<td>0.76</td>
<td>20.10</td>
<td>n/a</td>
<td>n/a</td>
<td>715.6</td>
<td>229.6</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td>0.84</td>
<td>0.76</td>
<td>20.10</td>
<td>714.0</td>
<td>0.52</td>
<td>716.2</td>
<td>229.1</td>
</tr>
</tbody>
</table>

* values based on b.m.t.

Figure C-13. Stress-strain curve for 1.00 mm thick G550 steel (coated)

Figure C-14. Stress-strain curve for 0.75 mm thick G550 steel (coated)
Appendix D. Tests results for combined shear-tension TOX tests

APPENDIX D
TEST RESULTS FOR COMBINED SHEAR-TENSION TOX TESTS

Results for each of the 40 small-scale tests, carried out to study the behaviour of the TOX press-joints used in the hybrid steel deck in combined shear and tension, are presented in this Appendix. A description of the test set-up as well as a discussion of the test results is given in Chapter 5.

Slip between the web and the base panel was measured on both sides, viz. north and south, using 15 mm linear potentiometers (LP’s) glued onto the ribs of the base panel and metal targets glued onto the lower flange of the webs.

Separation of the connector on the north and south side was obtained by measuring the lateral displacement of the TOX on the die side (web) as well as on the punch side (base panel) and subtracting the readings from each other since a direct measurement of the separation was not possible. The 15 mm LP’s were therefore directly attached to the RHS-frame to provide an absolute reference point. Measuring displacement on both sides of the TOX ensured that uplift and possible bending of the base panel was eliminated when determining separation of the connector.

Each specimen was given a distinct numeric code, e.g. 1007060-2, determined from basic information. The initial three numbers characterize the nominal thickness (in millimetres times 100) of the base panel, e.g. 1.00 mm, followed by two numbers which define the nominal thickness (in millimetres times 100) of the web, e.g. 0.70 mm. The subsequent two numbers represent the loading angle, e.g. 60 degree, and the final number, separated by a dash, states the specimen number, e.g. 2.
Appendix D. Tests results for combined shear-tension TOX tests

0.75 mm base panel, 0.70 mm webs, 0 degree

Figure D-1. Specimen 0757000-1

Figure D-2. Specimen 0757000-2
Appendix D. Tests results for combined shear-tension TOX tests

Figure D-3. Specimen 0757060-1

Figure D-4. Specimen 0757060-2

0.75 mm base panel, 0.70 mm webs, 60 degree
Appendix D. Tests results for combined shear-tension TOX tests

0.75 mm base panel, 0.70 mm webs, 60 degree

Figure D-5. Specimen 0757069-1

Figure D-6. Specimen 0757069-2
Appendix D. Tests results for combined shear-tension TOX tests

Figure D-7. Specimen 0757082-1

Figure D-8. Specimen 0757082-2
Appendix D. Tests results for combined shear-tension TOX tests

0.75 mm base panel, 0.70 mm webs, 90 degree

Figure D-9. Specimen 0757090-1

Figure D-10. Specimen 0757090-2
Appendix D. Tests results for combined shear-tension TOX tests

0.75 mm base panel, 0.90 mm webs, 0 degree

Figure D-11. Specimen 0759000-1

Figure D-12. Specimen 0759000-2
Appendix D. Tests results for combined shear-tension TOX tests

Figure D-13. Specimen 0759060-1

Figure D-14. Specimen 0759060-2
Appendix D. Tests results for combined shear-tension TOX tests

Figure D-15. Specimen 0759069-1

Figure D-16. Specimen 0759069-2
Appendix D. Test results for combined shear-tension TOX tests

Figure D-17. Specimen 0759082-1

Figure D-18. Specimen 0759082-2
Appendix D. Tests results for combined shear-tension TOX tests

0.75 mm base panel, 0.90 mm webs, 90 degree

Figure D-19. Specimen 0759090-1

Figure D-20. Specimen 0759090-2
Appendix D. Tests results for combined shear-tension TOX tests

1.00 mm base panel, 0.70 mm webs, 0 degree

Figure D-21. Specimen 1007000-1

Figure D-22. Specimen 1007000-2
Appendix D. Tests results for combined shear-tension TOX tests

1.00 mm base panel, 0.70 mm webs, 60 degree

Figure D-23. Specimen 1007060-1

Figure D-24. Specimen 1007060-2
Appendix D. Tests results for combined shear-tension TOX tests

Figure D-25. Specimen 1007069-1

Figure D-26. Specimen 1007069-2

1.00 mm base panel, 0.70 mm webs, 69 degree
Appendix D. Tests results for combined shear-tension TOX tests

1.00 mm base panel, 0.70 mm webs, 82 degree

Figure D-27. Specimen 1007082-1
Figure D-28. Specimen 1007082-2
Appendix D. Tests results for combined shear-tension TOX tests

Figure D-29. Specimen 1007090-1

Figure D-30. Specimen 1007090-2

1.00 mm base panel, 0.70 mm webs, 90 degree
Appendix D. Test results for combined shear-tension TOX tests

1.00 mm base panel, 0.90 mm webs, 0 degree

Figure D-31. Specimen 1009000-1

Figure D-32. Specimen 1009000-2
Appendix D. Tests results for combined shear-tension TOX tests

1.00 mm base panel, 0.90 mm webs, 60 degree

Figure D-33. Specimen 1009060-1
Figure D-34. Specimen 1009060-2
Appendix D. Tests results for combined shear-tension TOX tests

Figure D-35. Specimen 1009068-1

Figure D-36. Specimen 1009068-2
Appendix D. Tests results for combined shear-tension TOX tests

1.00 mm base panel, 0.90 mm webs, 82 degree

Figure D-37. Specimen 1009082-1
Figure D-38. Specimen 1009082-2
Appendix D. Tests results for combined shear-tension TOX tests

1.00 mm base panel, 0.90 mm webs, 90 degree

Figure D-39. Specimen 1009090-1

Figure D-40. Specimen 1009090-2
Results for each of the 18 full scale flexural tests, carried out to study the effect of TOX connector spacing in the base panel on the behaviour of the hybrid steel deck, are presented in this Appendix. A description of the test set-up as well as a discussion of the test results is given in Chapter 6.

Linear potentiometers (LP’s) with a range of 100 mm were used to measure vertical deflection near the loading points (800 mm east and 800 mm west from midspan) at three distinctive points across the base panel, and on the north side at mid-span. Longitudinal slip between the base panel and the web was measured on the north side of both shear spans using 15 mm LP’s hot-glued onto the ribs of the base panel and targets hot-glued between the connectors onto the lower flange of the web. Additionally, up to five strain gauges (T-N, N, M, S, and T-S) were attached to the bottom of the base panel at mid-span.

For each specimen, the longitudinal slip measurements of the 15 mm LP’s are presented (the location of each measurement point is stated in the boxes on the graph and defines the distance from the end of the panel in millimetres) as well as the slip distribution along the east and/or west side of the panels at certain applied loads (stated above each line in kilo-Newtons). The data of the slip distribution was extracted from the slip measurements.

A distinct alpha-numeric code, e.g. 10070100-A, determined from basic information was given to each specimen. The initial three numbers characterize the nominal thickness of the base panel (in millimetres times 100), e.g. 1.00 mm, followed by the nominal thickness of the web (in millimetres times 100), e.g. 0.70 mm. The subsequent two to three numbers represent the constant connector spacing of the base panel in millimetres, e.g. 100 mm. The final letter, separated by a dash, states the configuration, e.g. end anchorage (A) or free (F).
Appendix E. Test results for base panel shear tests

Figure E-1. Deflection and strain measurements of specimen 0757060-A

Figure E-1. Deflection and strain measurements of specimen 0757060-A
Appendix E. Test results for base panel shear tests

Figure E-2. Slip distribution of specimen 0757060-A

Figure E-3. Slip measurements of specimen 0757060-A
Appendix E. Test results for base panel shear tests

Test results of specimen 0757080-A

Figure E-4. Deflection and strain measurements of specimen 0757080-A
Figure E-5. Slip distribution of specimen 0757080-A

Figure E-6. Slip measurements of specimen 0757080-A
Figure E-7. Deflection and strain measurements of specimen 07570100-A
Appendix E. Tests results for base panel shear tests

Figure E-8. Slip distribution of specimen 07570100-A

Figure E-9. Slip measurements of specimen 07570100-A
Appendix E. Test results for base panel shear tests

Test results of specimen 07570120-A

Figure E-10. Deflection and strain measurements of specimen 07570120-A
Appendix E. Tests results for base panel shear tests

Figure E-11. Slip distribution of specimen 07570120-A

Figure E-12. Slip measurements of specimen 07570120-A
Appendix E. Test results for base panel shear tests

Figure E-13. Deflection and strain measurements of specimen 0757080-F
Appendix E. Tests results for base panel shear tests

Figure E-14. Slip distribution of specimen 0757080-F

Figure E-15. Slip measurements of specimen 0757080-F
Appendix E. Test results for base panel shear tests

Figure E-16. Deflection and strain measurements of specimen 07570100-F

Test results of specimen 07570100-F.
Appendix E. Test results for base panel shear tests

Figure E-17. Slip distribution of specimen 07570100-F

Figure E-18. Slip measurements of specimen 07570100-F
Figure E-19. Deflection and strain measurements of specimen 07570120-F
Figure E-20. Slip distribution of specimen 07570120-F

Figure E-21. Slip measurements of specimen 07570120-F
Figure E-22. Deflection and strain measurements of specimen 1007055-A

Test results for base panel shear tests
Appendix E. Test results for base panel shear tests

Figure E-23. Slip distribution of specimen 10070055-A

Figure E-24. Slip measurements of specimen 10070055-A
Appendix E. Tests results for base panel shear tests

Figure E-25. Deflection and strain measurements of specimen 1007085-A
Appendix E. Tests results for base panel shear tests

Figure E-26. Slip distribution of specimen 10070085-A

Figure E-27. Slip measurements of specimen 10070085-A
Appendix E. Test results for base panel shear tests

Test results of specimen 1007100-A

Figure E-28. Deflection and strain measurements of specimen 1007100-A
Figure E-29. Slip distribution of specimen 10070100-A

Figure E-30. Slip measurements of specimen 10070100-A
Test results of specimen 1007120-A

Figure E-31. Deflection and strain measurements of specimen 1007120-A
Figure E-32. Slip distribution of specimen 10070120-A

Figure E-33. Slip measurements of specimen 10070120-A
Appendix E. Test results for base panel shear tests

Figure E-34. Deflection and strain measurements of specimen 1007075-F

- North
- Centre
- South

Deflection 600mm East (mm) vs. Jack Load (kN)
Deflection 600mm West (mm) vs. Jack Load (kN)
Deflection Midspan (mm) vs. Jack Load (kN)
Strain at Midspan (%) vs. Jack Load (kN)
Appendix E. Tests results for base panel shear tests

Figure E-35. Slip distribution of specimen 10070075-F

Figure E-36. Slip measurements of specimen 10070075-F
Appendix E. Test results for base panel shear tests

Test results of specimen 10070100-F

Figure E-37. Deflection and strain measurements of specimen 10070100-F
Appendix E. Tests results for base panel shear tests

Figure E-38. Slip distribution of specimen 10070100-F

Figure E-39. Slip measurements of specimen 10070100-F
Appendix E. Test results for base panel shear tests

Figure E-40. Deflection and strain measurements of specimen 10070120-F

- North
- Centre
- South

Test results of specimen 10070120-F
Appendix E. Tests results for base panel shear tests

Figure E-41. Slip distribution of specimen 10070120-F

Figure E-42. Slip measurements of specimen 10070120-F
Appendix E. Test results for base panel shear tests

Test results of specimen 1009055-A

Figure E-43. Deflection and strain measurements of specimen 1009055-A
Appendix E. Tests results for base panel shear tests

Figure E-44. Slip distribution of specimen 10090055-A

Figure E-45. Slip measurements of specimen 10090055-A
Figure E-46. Deflection and strain measurements of specimen 10090075-F
Figure E-47. Slip distribution of specimen 10090075-F

Figure E-48. Slip measurements of specimen 10090075-F
Appendix E. Test results for base panel shear tests

Figure E-49. Deflection and strain measurements of specimen 10090100-F
Appendix E. Tests results for base panel shear tests

Figure E-50. Slip distribution of specimen 10090100-F

Figure E-51. Slip measurements of specimen 10090100-F
Figure E-52. Deflection and strain measurements of specimen 10090120-F.
Appendix E. Tests results for base panel shear tests

Figure E-53. Slip distribution of specimen 10090120-F

Figure E-54. Slip measurements of specimen 10090120-F
Appendix E. Test results for base panel shear tests

Test results of specimen 10090100-F (missing TOX)

Figure E-55. Deflection and strain measurements of specimen 10090100-F (missing TOX)
Figure E-56. Slip distribution of specimen 10090100-F (missing TOX)  
Figure E-57. Slip measurements of specimen 10090100-F (missing TOX)
APPENDIX F
PHOTOGRAMMETRIC DATA OF TOP PLATE BUCKLING TESTS

Digital close-range photogrammetry was employed to measure the top plate deformations before and during flexural testing of 18 specimens (refer to Chapter 8). Chapter 7 contains a detailed description of this non-contact measuring technique, including the test set-up used in this test series and a discussion on processing the obtained photogrammetric data. Longitudinal lines of each specimen before loading and just prior failure (last data set obtained just prior to buckling) of the top plate are presented in this Appendix. It also contains a 3D-contour plot of the post-buckled shape after failure. The post-buckled shape is then compared with the local deformations of the worst longitudinal line to verify if the overall plate buckle occurred at the same position as the most pronounced local buckle. Individual longitudinal lines were extracted and plotted for each line of targets on the north, middle and south sub-element of the top plate before loading and just prior failure. For each sub-element, a curve of best fit (in this case a parabola) was fitted to the worst longitudinal line to obtain the global curvature of the data. Subtracting this curvature from the data points enables the local variation of deformation to be determined, as can be seen in Chapter 8.4.2.

A distinct alpha-numeric code, determined from basic information, was given to each specimen. The initial two/three numbers characterize the nominal section height in millimetres, viz. 90 for a TD 90 section and 140 for a TD 140 section, followed by three numbers indicating the nominal thickness of the top plate (in millimetres times 100). The following letter represents the material grade of the top plate, where L stands for low-strength material (HA250), H for high-strength material (HA70T) and B for BLACKFORM material. The subsequent two numbers state the nominal thickness of the web (in millimetres times 100). The spacing of the top plate connectors at midspan is given in millimetres and separated by a dash. If the test was carried out with the original TOX connectors (instead of having the bolt replacement), the letter T was added after the connector spacing.
Appendix F. Photogrammetric data of top plate buckling tests

TD90: 2.50 mm top plate, 0.70 mm webs, 100 mm spacing

**Figure F-1. Longitudinal lines of specimen 90250B70-100 before loading**
Appendix F. Photogrammetric data of top plate buckling tests

Figure F-2. Longitudinal lines of specimen 90250B70-100 just prior failure

\[ y = 0.0000301499x^2 + 0.0006846072x - 169.8118103706 \]

\[ y = 0.0000320368x^2 + 0.0011359930x - 173.0366644233 \]

\[ y = 0.0000320511x^2 + 0.0012654690x - 172.8515169575 \]
Figure F-3. Post-buckled 3D-contour plot of specimen 90250B70-100

Figure F-4. Post-buckled shape vs. worst longitudinal line of specimen 90250B70-100
Appendix F. Photogrammetric data of top plate buckling tests

TD90: 2.50 mm top plate, 0.70 mm webs, 120 mm spacing (TOX)

\[ y = -0.0000005439x^2 - 0.0000685556x + 0.9216065758 \]

Figure F-5. Longitudinal lines of specimen 90250B70-120T before loading
Appendix F. Photogrammetric data of top plate buckling tests

Figure F-6. Longitudinal lines of specimen 90250B70-120T just prior failure
Figure F-7. Post-buckled 3D-contour plot of specimen 90250B70-120T

Figure F-8. Post-buckled shape vs. worst longitudinal line of specimen 90250B70-120T
Appendix F. Photogrammetric data of top plate buckling tests

TD90: 2.50 mm top plate, 0.70 mm webs, 140 mm spacing

Figure F-9. Longitudinal lines of specimen 90250B70-140 before loading
Appendix F. Photogrammetric data of top plate buckling tests

Figure F-10. Longitudinal lines of specimen 90250B70-140 just prior failure
Figure F-11. Post-buckled 3D-contour plot of specimen 90250B70-140

Figure F-12. Post-buckled shape vs. worst longitudinal line of specimen 90250B70-140
Appendix F. Photogrammetric data of top plate buckling tests

TD90: 2.50 mm top plate, 0.70 mm webs, 200 mm spacing

Figure F-13. Longitudinal lines of specimen 90250B70-200 before loading
Appendix F. Photogrammetric data of top plate buckling tests

Figure F-14. Longitudinal lines of specimen 90250B70-200 just prior failure
Appendix F. Photogrammetric data of top plate buckling tests

Figure F-15. Post-buckled 3D-contour plot of specimen 90250B70-200

Figure F-16. Post-buckled shape vs. worst longitudinal line of specimen 90250B70-200
Appendix F. Photogrammetric data of top plate buckling tests

TD90: 2.50 mm top plate, 0.70 mm webs, 280 mm spacing

Figure F-17. Longitudinal lines of specimen 90250B70-280 before loading
Figure F-18. Longitudinal lines of specimen 90250B70-280 just prior failure
Appendix F. Photogrammetric data of top plate buckling tests

Figure F-19. Post-buckled 3D-contour plot of specimen 90250B70-280

Figure F-20. Post-buckled shape vs. worst longitudinal line of specimen 90250B70-280
Appendix F. Photogrammetric data of top plate buckling tests

TD90: 2.50 mm top plate, 0.70 mm webs, 360 mm spacing

\[ y = -0.0000003171x^2 + 0.0012872054x + 1.7431233474 \]

\[ y = 0.0000002600x^2 + 0.0011039591x - 0.1139390909 \]

\[ y = 0.0000007911x^2 + 0.0009898807x - 1.3962161137 \]

Figure F-21. Longitudinal lines of specimen 90250B70-360 before loading
Appendix F. Photogrammetric data of top plate buckling tests

Figure F-22. Longitudinal lines of specimen 90250B70-360 just prior failure
Appendix F. Photogrammetric data of top plate buckling tests

Figure F-23. Post-buckled 3D-contour plot of specimen 90250B70-360

Figure F-24. Post-buckled shape vs. worst longitudinal line of specimen 90250B70-360
Appendix F. Photogrammetric data of top plate buckling tests

TD90: 1.60 mm top plate, 0.70 mm webs, 100 mm (TOX)

\[ y = 0.0000000492x^2 - 0.0008763401x + 2.1560074141 \]

\[ y = 0.0000005948x^2 - 0.0006929638x + 0.1095856742 \]

\[ y = 0.0000006504x^2 - 0.0003654652x + 1.1903572407 \]

Figure F-25. Longitudinal lines of specimen 90160B70-100T before loading
Figure F-26. Longitudinal lines of specimen 90160B70-100T just prior failure
Appendix F. Photogrammetric data of top plate buckling tests

Figure F-27. Post-buckled 3D-contour plot of specimen 90160B70-100T

Figure F-28. Post-buckled shape vs. worst longitudinal line of specimen 90160B70-100T
Appendix F. Photogrammetric data of top plate buckling tests

TD90: 1.60 mm top plate, 0.70 mm webs, 100 mm

Figure F-29. Longitudinal lines of specimen 90160B70-100 before loading
Figure F-30. Longitudinal lines of specimen 90160B70-100 just prior failure
Figure F-31. Post-buckled 3D-contour plot of specimen 90160B70-100

Figure F-32. Post-buckled shape vs. worst longitudinal line of specimen 90160B70-100
Appendix F. Photogrammetric data of top plate buckling tests

TD90: 1.60 mm top plate, 0.70 mm webs, 280 mm

Figure F-33. Longitudinal lines of specimen 90160B70-280 before loading
Appendix F. Photogrammetric data of top plate buckling tests

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Figure F-34. Longitudinal lines of specimen 90160B70-280 just prior failure
Appendix F. Photogrammetric data of top plate buckling tests

Figure F-35. Post-buckled 3D-contour plot of specimen 90160B70-280

Figure F-36. Post-buckled shape vs. worst longitudinal line of specimen 90160B70-280
Appendix F. Photogrammetric data of top plate buckling tests

TD140: 1.85 mm top plate (HA250), 0.70 mm webs, 100 mm spacing

Figure F-37. Longitudinal lines of specimen 140185L70-100 before loading
Figure F-38. Longitudinal lines of specimen 140185L70-100 just prior failure
Appendix F. Photogrammetric data of top plate buckling tests

Figure F-39. Post-buckled 3D-contour plot of specimen 140185L70-100

Figure F-40. Post-buckled shape vs. worst longitudinal line of specimen 140185L70-100
Appendix F. Photogrammetric data of top plate buckling tests

TD140: 1.85 mm top plate (HA250), 0.70 mm webs, 200 mm spacing

Figure F-41. Longitudinal lines of specimen 140185L70-200 before loading
Appendix F. Photogrammetric data of top plate buckling tests

Figure F-42. Longitudinal lines of specimen 140185L70-200 just prior failure
Appendix F. Photogrammetric data of top plate buckling tests

Figure F-43. Post-buckled 3D-contour plot of specimen 140185L70-200

Figure F-44. Post-buckled shape vs. worst longitudinal line of specimen 140185L70-200
Appendix F. Photogrammetric data of top plate buckling tests

TD140: 1.85 mm top plate (HA250), 0.90 mm webs, 200 mm spacing

Figure F-45. Longitudinal lines of specimen 140185L90-200 before loading
Appendix F. Photogrammetric data of top plate buckling tests

Figure F-46. Longitudinal lines of specimen 140185L90-200 just prior failure
Figure F-47. Post-buckled 3D-contour plot of specimen 140185L90-200

Figure F-48. Post-buckled shape vs. worst longitudinal line of specimen 140185L90-200
Appendix F. Photogrammetric data of top plate buckling tests

TD140: 1.85 mm top plate (HA250), 0.90 mm webs, 300 mm spacing

Figure F-49. Longitudinal lines of specimen 140185L90-300 before loading
Appendix F. Photogrammetric data of top plate buckling tests

Figure F-50. Longitudinal lines of specimen 140185L90-300 just prior failure
Appendix F. Photogrammetric data of top plate buckling tests

Figure F-51. Post-buckled 3D-contour plot of specimen 140185L90-300

Figure F-52. Post-buckled shape vs. worst longitudinal line of specimen 140185L90-300
Appendix F. Photogrammetric data of top plate buckling tests

TD140: 1.90 mm top plate (HA70T), 0.70 mm webs, 100 mm spacing

Figure F-53. Longitudinal lines of specimen 140190H70-100 before loading
Figure F-54. Longitudinal lines of specimen 140190H70-100 just prior failure
Figure F-55. Post-buckled 3D-contour plot of specimen 140190H70-100

Figure F-56. Post-buckled shape vs. worst longitudinal line of specimen 140190H70-100
Appendix F. Photogrammetric data of top plate buckling tests

TD140: 1.90 mm top plate (HA70T), 0.70 mm webs, 200 mm spacing

Figure F-57. Longitudinal lines of specimen 140190H70-200 before loading
Figure F-58. Longitudinal lines of specimen 140190H70-200 just prior failure
Figure F-59. Post-buckled 3D-contour plot of specimen 140190H70-200

Figure F-60. Post-buckled shape vs. worst longitudinal line of specimen 140190H70-200
Appendix F. Photogrammetric data of top plate buckling tests

TD140: 1.90 mm top plate (HA70T), 0.90 mm webs, 200 mm spacing

Figure F-61. Longitudinal lines of specimen 140190H90-200 before loading
Figure F-62. Longitudinal lines of specimen 140190H90-200 just prior failure
Figure F-63. Post-buckled 3D-contour plot of specimen 140190H90-100

Figure F-64. Post-buckled shape vs. worst longitudinal line of specimen 140190H90-200
Appendix F. Photogrammetric data of top plate buckling tests

TD140: 1.90 mm top plate (HA70T), 0.90 mm webs, 300 mm spacing

Figure F-65. Longitudinal lines of specimen 140190H90-300 before loading
Appendix F. Photogrammetric data of top plate buckling tests

Figure F-66. Longitudinal lines of specimen 140190H90-300 just prior failure
Figure F-67. Post-buckled 3D-contour plot of specimen 140190H90-300

Figure F-68. Post-buckled shape vs. worst longitudinal line of specimen 140190H90-300
The exact shape of the top plates used in 18 full scale flexural tests (refer to Chapter 8) were obtained by making casts of the outer surface of the plates using CIVILCAP material. Generally, two casts (400 mm east and 400 mm west from midspan) were made for each specimen of this test series, as can be seen in Figure G-1. The traced outline of each cast, including the relative position of the TOX connector (indicated with arrows) is presented in this Appendix.

Figure G-1. Casts of individual top plates
Figure G-2. Traced outlines of 2.50 mm thick top plates (70T, 100, 120T, 140 mm spacing)
Figure G-3. Traced outlines of 2.50 mm thick top plates (200, 280, 360 mm spacing)
Figure G-4. Traced outlines of 1.60 mm thick top plates (100T, 100, 280 mm spacing)
Figure G-5. Traced outlines of 1.85 mm thick top plates (100, 200 mm spacing)
Figure G-6. Traced outlines of 1.85 mm thick top plates (200, 300 mm spacing)
Figure G-7. Traced outlines of 1.90 mm thick top plates (100, 200 mm spacing)
Figure G-8. Traced outlines of 1.90 mm thick top plates (200, 300 mm spacing)
APPENDIX H
TESTS RESULTS FOR TOP PLATE BUCKLING TESTS

Results for each of the 18 full scale flexural tests, carried out to study the top plate buckling behaviour of the hybrid steel deck, are presented in this Appendix. A description of the test set-up as well as a discussion of the test results is given in Chapter 8.

Linear potentiometers (LP's) with a range of 100 mm were used to measure vertical deflection at three distinctive points across the base panel at midspan and on the north/south side 800 mm east and 800 mm west from midspan. All specimens had at least three strain gauges attached to the outside surface of the top plate at midspan. Generally, one strain gauge was centrally placed on the plate between the longitudinal stiffener and one strain gauge above the web-flange intersection on each edge plate element (north and south). The strain gauges on the edge plate elements were centrally placed between TOX connectors to minimise the effect of potential stress concentrations around the connectors. For most specimens, additional strain gauges were attached to the upper and lower part of the corrugated web, as well as to the bottom of the base panel at mid-span. The data can be used to obtain information about the strain distribution over the depth of the section during bending, as well as to estimate the stresses achieved in the top plate. The exact strain gauge locations of each specimen are given in the corresponding load-strain diagrams.

A distinct alpha-numeric code, determined from basic information, was given to each specimen. The initial two/three numbers characterize the nominal section height in millimetres, viz. 90 for a TD 90 section and 140 for a TD 140 section, followed by three numbers indicating the nominal thickness of the top plate (in millimetres times 100). The following letter represents the material grade of the top plate, where L stands for low-strength material (HA250), H for high-strength material (HA70T) and B for BLACKFORM material. The subsequent two numbers state the nominal thickness of the web (in millimetres times 100). The spacing of the top plate connectors at midspan is given in millimetres and separated by a dash. If the test was carried out with the original TOX connectors (instead of having the bolt replacement), the letter T was added after the connector spacing.
Appendix H. Tests results for top plate buckling tests

TD90: 2.50 mm top plate, 0.70 mm webs, 70 mm spacing (TOX)

Figure H-1. Deflection and strain measurements of specimen 90250B70-70T
Appendix H. Tests results for top plate buckling tests

TD90: 2.50 mm top plate, 0.70 mm webs, 100 mm spacing

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Figure H-2. Deflection and strain measurements of specimen 90250B70-100

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Appendix H. Tests results for top plate buckling tests

TD90: 2.50 mm top plate, 0.70 mm webs, 120 mm spacing (TOX)

Figure H-3. Deflection and strain measurements of specimen 90250B70-120T
Appendix H. Tests results for top plate buckling tests

TD90: 2.50 mm top plate, 0.70 mm webs, 140 mm spacing

Figure H-4. Deflection and strain measurements of specimen 90250B70-140
Appendix H. Tests results for top plate buckling tests

TD90: 2.50 mm top plate, 0.70 mm webs, 200 mm spacing

Figure H-5. Deflection and strain measurements of specimen 90250B70-200
Appendix H. Test results for top plate buckling tests

Figure H-6. Deflection and strain measurements of specimen 90250B70-280
Appendix H. Tests results for top plate buckling tests

TD90: 2.50 mm top plate, 0.70 mm webs, 360 mm spacing

Figure H-7. Deflection and strain measurements of specimen 90250B70-360
Appendix H. Tests results for top plate buckling tests

TD90: 1.60 mm top plate, 0.70 mm webs, 100 mm spacing (TOX)

Figure H-8. Deflection and strain measurements of specimen 90160B70-100T

Figure H-8. Deflection and strain measurements of specimen 90160B70-100T
Appendix H. Tests results for top plate buckling tests

TD90: 1.60 mm top plate, 0.70 mm webs, 100 mm spacing

Figure H-9. Deflection and strain measurements of specimen 90160B70-100

- Load (kN)
- Deflection (mm)
- Stress at Midspan (%)
- Jack Load (kN)

Figure H-9. Deflection and strain measurements of specimen 90160B70-100
Appendix H. Tests results for top plate buckling tests

TD90: 1.60 mm top plate, 0.70 mm webs, 280 mm spacing

Figure H-10. Deflection and strain measurements of specimen 90160B70-280

![Graphs showing deflection and strain measurements for specimen 90160B70-280.](image-url)
Figure H-11. Deflection and strain measurements of specimen 140185L70-100
Appendix H. Tests results for top plate buckling tests

TD140: 1.85 mm top plate (HA250), 0.70 mm webs, 200 mm spacing

Figure H-12. Deflection and strain measurements of specimen 140185L70-200
Appendix H. Tests results for top plate buckling tests

TD140: 1.85 mm top plate (HA250), 0.90 mm webs, 200 mm spacing

Figure H-13. Deflection and strain measurements of specimen 140185L90-200
Figure H-14. Deflection and strain measurements of specimen 140185L90-300
Appendix H. Test results for top plate buckling

TD140: 1.90 mm top plate (HA70T), 0.70 mm webs, 100 mm spacing

Figure H-15. Deflection and strain measurements of specimen 140190H70-100
Figure H-16. Deflection and strain measurements of specimen 140190H70-200
Appendix H. Tests results for top plate buckling tests

TD140: 1.90 mm top plate (HA70T), 0.90 mm webs, 200 mm spacing

Figure H-17. Deflection and strain measurements of specimen 140190H90-200
Appendix H. Tests results for top plate buckling tests

Figure H-18. Deflection and strain measurements of specimen 140190H90-300

Deflection Midspan (mm)

Deflection 800mm East (mm)

Deflection 800mm West (mm)

Strain at Midspan (%)

Deflection 800mm West (mm)